Highway Surveying Manual

M 22-97

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Washington State Department of Transportation
Environmental and Engineering Service Center
Design Office

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Foreword

This Highway Surveying Manual presents surveyors' methods and departmental rules that apply to highway surveying operations of the Washington State Department of Transportation. The manual is intended to help standardize surveying practices throughout the Department and to be a useful tool for WSDOT survey crews.

Updating the manual is a continuing process and revisions are issued periodically. Questions, observations, and recommendations are invited. The next page is provided to encourage comments and assure their prompt delivery. Use copies of it to transmit comments and attachments, such as marked copies of manual pages. For clarification of the content of the manual, contact the Computer Aided Engineering Branch in the Olympia Service Center.

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Subject: *Highway Surveying Manual* comment

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Survey Crew

Survey crews in the Washington State Department of Transportation (WSDOT) perform many important functions. They collect field data for the design and construction of highways. They establish the center lines of proposed highways, they reestablish the center lines of existing highways, and they identify the property the department controls. They lay out structures and do all the required staking for construction. All this is done to an appropriate level of accuracy for efficient utilization of time, manpower, and equipment.

The survey crew should have a thorough knowledge of the *Highway Engineering Field Formulas*, the Standard Specifications, the Standard Plans, the *Construction Manual*, portions of the *Design Manual*, contract plans, special provisions, and this *Highway Surveying Manual*.

WSDOT survey crews usually consist of four people, depending upon the crew's assignment and other considerations.

The survey crew members should regularly review the previously listed publications paying particular attention to section 1-5 of the *Construction Manual* (M 41-01).

Section 1-5 discusses surveying errors. These are not errors by definition, as you will learn later in this manual, but are rather mistakes or blunders. Some of these problems arise due to inadequacy of control points by loss or destruction, or they were never set. Other causes may be incorrect plans, incorrect interpretation of the plans, overlooking a detail in the plans, and miscommunication. There are probably more. The best way to avoid mistakes or blunders is to check your work.

The party chief, a Transportation Engineer 2, is the person who supervises the crew. The party chief directs the crew, does the planning on how to complete the day's assignment, and usually records the work. However, the record keeping may be delegated to other crew members.

The instrument operator, a Transportation Engineer 1 or Transportation Technician 3, is responsible for the efficient setup and operation of all the survey instruments. In the party chief's absence, he directs the crew's activities.

Other members of the crew are Transportation Technicians 1 and 2. Their duties are to set up signs and control the traffic, make measurements with a tape or chain, set up reflector prisms on tripods or staffs, hold the leveling rod, and perform other tasks as directed by the party chief. The Tech 1's and 2's should have taken the WSDOT basic survey training course. Whenever possible, the party chief should rotate the duties of the crew members so that each receives varied training and experience. This is to everyone's benefit and professional growth.

The party chief reports to the project engineer or the project engineer's delegate. The delegate would be the assistant project engineer, the chief of parties, or the project inspector.

The party chief is responsible for keeping the daily diary showing arrival and departure times, recording a brief description of the day's work and unusual conditions, events, or circumstances.

Responsibilities

Responsibilities to the Public

WSDOT surveyors are seen by the public as representatives of the department. If they are knowledgable concerning the department's policies and the project at hand; and if they answer inquiries and requests honestly, openly, and accurately; then the department will be well represented. Courtesy, patience, attentive listening, accuracy, truthfulness, and even legal driving practices are all responsibilities of a surveyor when in the public eye as a representative of the department and the state.

Responsibilities to the Contractor

On a construction project, the crew reports to the project engineer or the project engineer's delegate. The crew should also have a good working relationship with the contractor. Section 1-05 of the Standard Specifications basically states that the engineer will set stakes one time and that the contractor will provide safe and sufficient facilities for setting points and elevations. The timing of contractor requests for stakes is addressed in the *Standard Specifications*. Identify all stakes as provided elsewhere in this manual.

Contractor surveying is becoming commonplace. The special provisions specify the criteria for precision and set precedence in case of dispute between the contractor and the state.

Relations with Other Departments and Agencies

Federal Highway Administration

FHWA engineers make final inspections of construction on all projects funded wholly or in part with federal-aid funds to verify substantial compliance with the approved program. In addition to these final inspections, they are required to make intermediate inspections and reviews on interstate and other preselected projects. Notes of important comments made by the FHWA inspector should be made and passed to the project engineer but, when the FHWA inspector is accompanied by the project engineer, district officials and/or headquarters officials, notes will be taken by them. During these inspections, all department personnel should cooperate with representatives of the FHWA in all matters pertaining to the project.

Other Federal, State, and Local Agencies

There are several state laws having to do with the relationship between WSDOT and other agencies as regards surveying activities. The Department of Natural Resources (DNR) maintains a public record of the surveys performed in the state of Washington and Geographic Services provides data to the WSDOT Geographic Information System (GIS). So, in addition to providing survey information to Geographic Services, WSDOT surveyors must, as required by state law, provide survey information to DNR and either to the county engineer or auditor. In addition, state law requires that a permit be obtained from DNR before any existing monument may be disturbed. Chapter 9 of this manual, and Chapters 1440 and 1450 of the *Design Manual,* describe these requirements in detail.

Safety

While the party chief is charged with the responsibility of providing safety leadership, personal safety should be a primary concern for all crew members. All must demonstrate through actions and words a commitment to safety.

Each member of the crew must be familiar with section 1-1.9, Safety and Accidents, of the *Construction Manual* (M 41-01) and the *Safety Manual*, M 75-01.

When working in traffic areas the survey crew must have all signing in conformance with the *Traffic Control Guidelines for Survey Operations* (M 55-02). When the survey crew is working under the protection of the contractor's traffic control, additional signing may not be necessary. Communicate with the contractor that the crew will be working within the traffic control setup. Be aware that the traffic control setup may change as work progresses and additional signing or a complete new setup may be required.

The party chief must possess a first aid card and the crew rig must contain a first aid kit.

Take every precaution and use common sense when working adjacent to or stepping into the traveled way. Stay visible and do not park or stand behind equipment. Always be aware of construction activities and the path of construction equipment.

For future safety, set alignment monuments on an offset line and, if possible, relocate existing center line monuments to an offset line.

Be completely prepared prior to setup. This will minimize the crews exposure to hazardous situations and keep the delays to the public as short as possible. However, never sacrifice safety to reduce the time to complete a task.

Complete attention to all safety rules will reduce accidents. This includes safety to the traveling public as well as to the contractor's personnel and state employees. Take extra safety precautions for traffic through and around construction operations.

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2
Equipment

Tools, supplies, and equipment will be required for nearly all surveying operations. Some tasks may require more specialized equipment. The equipment listed below will cover most situations:

Crew Equipment

Highway Surveying Manual
Engineering Field Tables
Traffic Control Guidelines for Survey Operations
STOP and SLOW paddles
first aid kit
SF-136 Vehicle Accident Checklist
SF-137 Vehicle Accident Report
total station with a minimum of two batteries
data collector
tripod legs (3 sets)
tribrachs (3)
tribrach adjusting hub
single mirrors with targets (2)
triple mirror with target
1-3 prism poles
rain covers for survey instruments
environmental bag for data collector
2-3 two-way radios with speaker mikes with 2 km range
theodolite and tripod
Philadelphia rods (2)
accurate rod level
100 m chain
30 m chain
15 m and 30 m fiberglass reinforced cloth tapes
range poles or pickets
automatic level and tripod
clinometer
Lenker rods (2)
fiberglass rod (7.5 m)
right angle prism
calculator with trig functions
thermometer
barometer
pliers
flashlight
first aid kit (16-unit minimum)
fire extinguisher
axe
4-6 kg sledge hammers (2) and extra handles
9 kg hammer
frost pins
shovel
pick
claw hammer
saw
files
small whisk broom
machete or brush axe
small and large Phillips and regular screwdrivers
chain clamps
jumper cables
tow cable
spray paint (blue, red, yellow, white, black, silver, fluorescent orange)
"Required" signs and standards
P.K. nails
plumb bob string
keel (blue, yellow)
plastic targets

300 mm targets
hub tacks
guineas
masking tape
200 mm, 300 mm, 460 mm, 600 mm hubs
stakes
lath
ribbon (red, white, blue, yellow)
railroad spikes
box nails and double headed nails
oil or silicone spray (WD-40)
traffic cones (600 mm)
dry lubricant (for instrument screws)
extra plumb bob points
lens cloth/tissue(s)
toilet paper

Personal Equipment

7.5 m pocket tape
plumb bob with sheath
hand level with sheath
hard hat
survey vest (reflectionized)
paper pad
plastic target
tack container
rain clothes
pocket knife
pencils
red ink pen

Care of Equipment

If the equipment you use is to function as it is intended, you must take proper care of it. All survey equipment used for horizontal and vertical control msut be checked against a standard. Keep records of survey instrument checks and calibrations in the project office.

Total Station, Theodolite, and Level

Total stations, theodolites, and levels are delicate and precise tools and should be handled accordingly.

When an instrument is being removed from its case or tripod, it should never be lifted by the scope or horizontal axis. The only exception is that a level may be lifted by the scope.

When setting up the instrument, the leveling screws should never be tightened more than is required to eliminate looseness. If you have to exert a great deal of force to loosen the screws, they were too tight. Overtightness (probably the most common abuse of instruments) sets up stresses which will warp vital parts, rupture fine threads, and will result in erratic readings. This also applies to all other threaded adjustment screws.

Never leave the instrument unattended when not in use. Never set an instrument behind a vehicle. If it is to be left near a vehicle, set it in view where the driver can easily see it.

Make sure the level is secured to the tripod before picking it up. When carrying the level through brush or trees or into a confined area, it should be carried under the arm with the head in a forward position where it can be seen. When the level is being carried on the tripod, the telescope axis should be tight enough to prevent movement by force of gravity, but loose enough to permit movement if the instrument is accidentally bumped. When transporting the instrument in the rig, the head should be removed from the tripod and carried in its case.

Total stations and theodolites should never be carried on the tripod. They should be boxed up for all moves. Total stations and theodolites transported in their boxes should have all motions free.

When the tripod is not in use, the cap should be fastened snugly. The threads on the tripod head should be kept clean. The graphite from a black stake pencil makes a good lubricant for the threads on the tripod and the instrument. Use a dry lubricant if available, not oil or grease which will trap grit and result in unnecessary wear on the threads.

A plastic cover should be placed over the instrument when it is not being used or if it is raining. A hot sun will affect the instrument. Use shade to protect the instrument. Use an umbrella if you must measure in the rain. If the instrument does become wet, air dry it (out of its case) overnight.

Hand Level, Clinometer, and Other Small Instruments

Hand levels, clinometers, and right-angle prisms should be kept in their cases when not in use. Right-angle prisms should be closed when not in use. Hand levels should be taken apart and cleaned when they become wet or dirty. Check hand levels and clinometers for accuracy periodically when new,and if dropped.

To clean a lens, use a lens cloth or tissue, not a cotton swab. Remove dried concrete with a piece of copper wire.

Level Rod and Prism Pole

The leveling rod and prism pole must be properly maintained if good quality data are to be collected.

The rodman should guard the rod against physical damage and protect it from the effects of exposure to the outdoor environment. Rods should be placed in a dry place at night if they have been used in wet conditions. Rods should be stored in a vertical position or lying fully supported in a horizontal position. Do not use the rod for a pole vault or to beat down brush. Keep fingers off the face of the level rod as much as possible. Excessive handling will wear off the numbers. Striking the rod against rocks, trees, signs, vehicles, etc., can chip the special material used for rod scales, making it difficult to continue observing precisely and efficiently. Keep the screws on the level rod hardware snug, but do not overtighten. The tape on the Lenker rod should be checked frequently. If a tear develops, the tape should be replaced. Check multisection rods occasionally and adjust as needed.

Steel Chain

- Never pull a chain around a post, stake, or other sharp object.
- Do not allow the chain to be run over by any vehicles.
- Be very careful to avoid kinks or sharp bends in the chain.
- Do not hold the chain by bending it around your hand or standing on it.
- Use the chaining clamps.

When the chain gets wet, it should be treated with a rust preventive spray. A chain on a reel should be wound backwards until the chain is loose and then be sprayed. The chain does not have to be dry since the spray will displace the water. If the chain is muddy, wash it before treatment. After a while, the spray will cause the chain to get oily and hard to read; therefore, periodically wipe it with a clean dry cloth. A chain cared for in this manner will last for many years, even when it is used daily in coastal areas.

Checking Instruments

Checklist

- ☐ Put on tripod and let it sit until its temperature nears that of surrounding air
- ☐ Run foot screws up and down
- ☐ Check tangents and foot screws for smoothness over entire travel
- ☐ Check tribrach locks and spring plate for secure hold down
- ☐ Rotate scope through standards several times
- ☐ Rotate spindle several times
- ☐ Check focus/parallax at 2-8 m and at 50-90 m
- ☐ Check objective/eyepiece for “halo, haze”
- ☐ Advance horizontal circle — check for “spots”
- ☐ Move micrometer control — check for “spots”
- ☐ Plate bubble check/adjust
- ☐ Horizontal/vertical collimation routines
- ☐ Turn sets of horizontal/vertical angles in 4 quadrants

Adjustment of Equipment

In order to obtain consistent results from your survey instruments they must be kept in good adjustment. Some adjustments must be done in a shop by trained personnel. Total stations should be sent in once a year for cleaning and adjustment. However, most common adjustments can be taken care of in the field. The following data is generally applicable to most instruments. For individual brands and types consult the manufacturer’s recommendations, if available.

Preadjustment steps

Before you decide to adjust your instrument you should test it in the proper manner to make sure adjustment is necessary. The following steps are recommended.

Choose a cloudy day if possible. Heat waves make it nearly impossible to obtain accurate readings.

See that the tripod is in good condition. Tighten the shoes and all other hardware.

Set up on firm ground where you can see at least 60 m in each direction and where the ground is fairly level. Do not set up on asphalt on a warm day as the instrument may settle during the test. Spread the legs uniformly and set them firmly in the ground. Set up so that the tripod is nearly level. If a conventional tripod is used, loosen and tighten all three tripod hinge screws to make sure they are not in tension. If the ball type is used, make sure it is not loose at the socket. If it is, there is an adjustment to tighten it with.

Level up the instrument. Then loosen the level screws and then relevel to assure there is no tension. Do not tighten leveling screws more than just snug. Allow the instrument to set up for a few minutes to allow it to adjust to the outside temperature.

Testing and Adjusting the Optical Plummet

The method of adjusting an optical plummet involves moving the adjusting screws. The configuration of adjusting screws is not standard among the manufacturers. The adjusting screws are either capstan screws with four holes in the caps that allow you to insert an adjusting pin, or slotted screws that can be turned with a screwdriver. Whatever configuration the adjusting screws, the process of adjusting the optical plummet is as follows:

Mount one tribrach on the tripod with the adjusting adapter in place. Mark a point on the ceiling above the tripod. Place the tribrach to be adjusted on the adapter in an inverted position. Using the lower tribrach, put the cross line on a point directly on the ceiling point. Rotate the upper tribrach 180°. Adjust one half of the error with the adjusting screws. The other half of the error is adjusted with the lower tribrach (foot) screws. Check in each quadrant and adjust as necessary.

The above procedure is for use with a special adapter and two tribrachs. Other optical plummet adjustments (an alidade with a plumb bob or without a special adapter) are out lined in the instrument manual.

Testing and Adjusting the Self-Leveling Level

There are several different makes in use throughout the state. You should consult the manufacturer's instructions before any adjustments are made. Most procedures are the same for all makes but the adjusting screws may be in different locations. Before adjusting the level, follow the preadjustment steps shown above. Now you are ready to make the tests to see if adjustment is required.

- **To see if the circular bubble stays centered when the level is rotated**

  This is very important because to get the maximum benefit out of the compensator mechanism the bubble should be in adjustment.

  TEST: Turn the telescope until it is parallel with two leveling screws. Center the bubble as close as you can. Turn the telescope 180° until it is parallel with the same leveling screws. The bubble should remain centered.

  ADJUSTMENT: You will have to check the individual instrument for the location of the adjusting screws. On the Zeiss level you remove the observation prism or unscrew the flat guard ring around the bubble. There are three screws for adjustment. Loosen each screw and then retighten each one evenly, finger tight. Repeat the test. If the bubble is not centered, bring it half way to center with the leveling screws. Bring it the rest of the way by tightening the most logical screw. Do not loosen any of them. Repeat the test. If it does not center, repeat the adjustment until the bubble remains centered when the level is rotated.

- **To make the line of sight level**

  These are the adjustments usually performed in the field. If you have dropped or otherwise damaged your instrument and cannot get it to come into proper adjustment do not try to take it apart and fix it. Send it to an authorized shop for repairs and adjustment.

  TEST: The peg method is described below:
  On fairly level ground set up the level. The test is best done on a cloudy cool day. Drive two stakes 30 m in each direction from the level. These are points A and C in Figure 2-1.

  Take a rod reading in each direction (with the same rod or a matched pair). Read the rod carefully estimating to the nearest mm 0.001 m. Compute the difference between the two readings. This is the true difference in elevation between the two stakes. In Figure 2-1 the true difference in elevations is 4.027 - 3.875 = 0.152.

  Next, set up about 6 m (10 percent of total distance) behind one of the stakes. See Figure 2-2.

  Read the rod on the near stake. In the example, 4.132 at A. The rod reading at C should equal the rod reading at A plus the true difference in elevation (4.132 + 0.152 = 4.284). The difference between this number and the actual rod reading at C is the error to be corrected by adjustment. Loosen the top (or bottom) capstan screw holding the cross hairs of the eyepiece, and tighten the bottom (or top) screw to move the horizontal hair up or down and give the required reading on the rod at C. Several trials may be necessary to get an exact reading. (*Caution*: One screw should be loosened before the other is tightened on older instruments to avoid breaking the cross hair!)

Testing and Adjusting the Hand Level and Clinometer

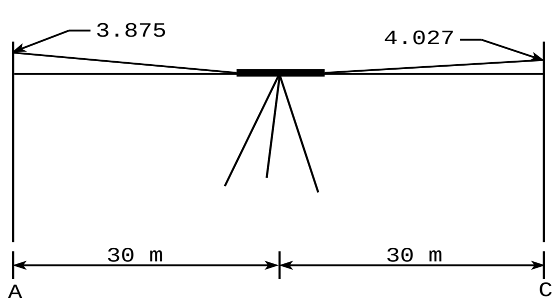
The hand level can be adjusted by the peg method. You should check it frequently. Peg the hand level at 15 m as follows:

- Set two stakes 15 m apart.
- Position yourself halfway between them to determine difference in elevation. Do not hold the hand level. Use a stand.
- Stand at one stake and shoot the other. Adjust the adjusting screw until the same difference in elevation is read. The screw is located in the objective end. You will have to remove the glass lens. You can also check your hand level against a level when this is convenient.

The clinometer should be pegged or checked against a level when you use it. Set the vernier at zero degrees and peg it the same as a hand level. If it requires adjustment there are adjusting screws on the level bubble tube.

Testing and Adjusting the Theodolite/Total Station

The tests and adjustments for the Wild T2 Theodolite should be carried out in the following order. Other makes and models of theodolites will have similar adjustments.



Peg Method, Setup #1
Adjusting the Self-Leveling Level
Figure 2-1

Adjust the movement of foot and micrometer screws. The foot and micrometer screws must work with moderate ease, but must not shake. If they turn too easily, the adjusting screw must be tightened to free the movement from backlash.

Adjust the horizontal level. Using the foot screws, bring the bubble of the circular level to center. This brings the circular level to approximate vertical position.

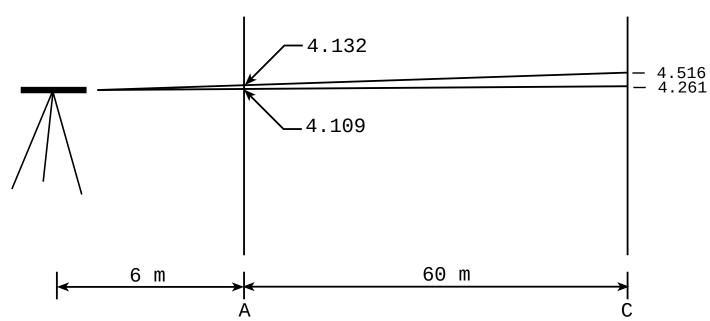
Turn the instrument until the horizontal level is parallel to a line joining two of the foot screws and center its bubble.

Turn the theodolite 180°. If the bubble is no longer centered, correct half the difference with the foot screws and the other half with the capstan head adjusting screw which is located under the left hand standard.

Turn the theodolite 90°, so that the level is perpendicular to the line used before, and center the bubble by using the third foot screw. This brings the vertical axis to verticality in space.

The above procedure is called "leveling the theodolite." If the horizontal level shows only a small discrepancy (1 or 2 divisions), it will be more convenient to note the position of the bubble than to correct the discrepancy. The leveling of the instrument is then done with reference to this position of the bubble.

Adjust the optical plummet. Accurately center the theodolite over a ground point by means of a plumb line inserted in the center fixing screw.



Peg Method, Setup #2
Adjusting the Self-Leveling Level
Figure 2-2

Make the vertical axis truly vertical by use of the horizontal axis.

Carefully remove the plumb line and determine whether the ground point appears exactly in the center of the small circle. If not, this can be corrected by means of the capstan head adjusting pen. This pen is inserted in the screw heads, and the screws are adjusted by first loosening the screw away from the direction in which the hole is to be moved and then tightening the screw on the same side as the desired direction of movement.

Adjust the collimation error. The horizontal collimation of the telescope can be adjusted by sliding the diaphragm (reticule). A screwdriver is needed for this purpose. To check this adjustment, point the telescope at a clearly defined point and take the reading of the horizontal circle.

Plunge the instrument and take the reading on the same point in the reversed or plunged position. These two readings should be exactly 180° apart; if not, the difference is equal to twice the collimation error.

Set the average value of the two drum readings on the micrometer drum, and turn the tangent screw up to the point of coincidence of the graduation lines. Move the diaphragm horizontally to obtain coincidence between the vertical line and the object pointed upon. For this purpose, there are three adjusting screws; one in a horizontal and two in a slant position. If the telescope direct (not reversed) requires moving the diaphragm to the left, loosen the two screws located in the slant position exactly the same extent to the right, and tighten the horizontal screw to the left.

Repeat these measurements and corrections until the collimation error is less than plus or minus 10 seconds. Check the vertical hair running it up or down a clearly defined point by using the tangent screw for the vertical motion. If the hair is truly vertical, it will follow a vertical course up or down. If further adjustment is needed, the diaphragm can be rotated slightly by turning the slanted screws in the opposite direction.

To adjust the collimation level, it is necessary, first to remove any inadmissible horizontal collimation error. (See above.) Then, bring the image of the vertical circle into view by turning the change over knob.

Bring the horizontal cross hairs onto a clearly defined object.

Level the collimation bubble by means of the bubble tangent screw and read the vertical circle.

Repeat the operation with the telescope in the reversed position. The sum of the two readings should be 360°. If not, collimation error or a so called index error of the vertical circle exists. If this error exists, compute the correct reading by adding or subtracting, depending on whether the total was more or less than 360°, one half of the direct reading error on the micrometer drum.

Bring the telescope to the direct position, exactly centered on the object.

Make the coordinates of the graduation lines with the tangent screw for the collimation level.

Bring it to the leveled position by means of the capstan head screws. If the collimation error has been sufficiently eliminated, the readings, for example, may be:

Telescope direct	87° 40′ 15″
Telescope reversed	92° 19′ 48″
	180° 00′ 03″

There is still a collimation error of 3 seconds, which needs no further elimination.

Testing and Adjusting the EDM

Most of the mechanism of an EDM is electronic and is internal. Adjustment of the electronics requires specialized equipment which is not available in the field. If the electronics of an EDM require adjustment or repair, it should only be done by a qualified technician with the proper equipment. However, there are some external adjustments that can be done in the field.

To calibrate an EDM, follow the procedures in NOAA Technical Memorandum NOS NGS-10.

WAC 332-130-100 requires annual calibration of distance measuring devices.

Set the instrument up on a calibrated test range and check the distances measured to the different distance monuments. If the measured distances fall within the

manufacturer's tolerances, note the measurement differences for correcting field measurements. If the measured distances exceed the tolerances, have the instrument checked for faulty electronics. Make sure that the errors are not due to faulty field procedures.
Some of these errors are:

- instrument or prism not exactly above points
- prism not matched to instrument
- atmospheric corrections not set in
- errors in height of instruments of reflectors
- instrument has not been given sufficient time to adjust to local temperature/pressure/humidity. Let the instrument warm up before measuring

To adjust the optical plummet of the tribrach, see Testing and Adjusting the Optical Plumment earlier in this chapter.

Before checking an EDM on a certified calibration base line, consider the following:

- Legs on the tripod tight?
- Tribrachs in adjustment?
- Barometer set correctly? (not corrected to sea level)
- Thermometer correct? (1° C can change reading by 1 PPM)
- Atmosphere corrections set in EDM?
- EDM given time to warm up and adjust to local atmospheric conditions?
- Prism matched to EDM?
- Prism and EDM at same HI? (If not, EDM with slope correction)

Testing and Adjusting the Tripod

Set up and level the instrument. Point to a well defined object over 150 m away. Twist the tripod head gently both ways and sight the object again. If the horizontal angle changes after setting, the tripod has developed looseness and the tripod must be adjusted, repaired or replaced.

See that no play exists at the junction of the wood and metal parts. If play exists, it should be eliminated by tightening the hexagonal nuts found on the foot plates and exteriors of the tripod head.

Check the working of the legs and, if necessary, tighten the clamping screws under the head of the tripod. When released from a horizontal position, the legs should fall to an angle of about 45° and remain there.

Use of Equipment

Setting up the tripod

The following should help avoid trouble. On level ground space the legs evenly. Try not to place a leg on the line of sight. Set the instrument at a height where both you and the rear chainman can reach it comfortably. On steep ground place two legs downhill and one uphill. Try to keep tripod head approximately level. Align the leveling screws with the line of sight. Place the lower clamp and tangent screws where you can easily reach them. When you are over the point, loosen the hinge clamps and retighten them on the old type tripods. This relieves tension. Do not tighten the leveling screws any more than enough to eliminate play. Tighten them evenly.

Turning Angles

When sighting a point, it is important that the final motion should always be made against the spring tension of the tangent screw. When repeating angles, always read the first angle and record it. If your average is very far off the first reading, turn the angle again. Always turn angles an even number of times. Take readings in both direct and reverse positions.

Self Compensating Level

Be particularly careful to avoid bumping or jarring this instrument. The compensating mechanism in some models is very delicate. Do not adjust the leveling screws after taking the initial reading. The compensating mechanism will take care of minor adjustments. (The head is not connected to the base except by the leveling screws.) It is possible to alter the reading by too much adjusting. Tap the level lightly to assure that the compensator is swinging freely.

The Lenker Rod

This rod allows the levelman to read the elevation directly. There are a few points to observe in the two basic methods for setting the rod.

One method is as follows:

- Loosen the clamp that holds the tape.
- On instructions from the levelman run the tape up or down until the levelman reads the last three numbers of the bench mark elevation.
- Lock the tape. Raise and lower the rod to make sure the tape does not slip. Have the levelman check his reading.

An alternate method is as follows:

- Set the rod on the bench mark and the levelman reads the rod.
- He tells the rodman what he reads.
- The rodman marks the spot on the edge of the rod by placing his thumb-nail at the reading.
- Adjust the tape until the last three numbers of the bench elevation are set at the mark.
- Lock the tape.
- Have the levelman check the reading.
- Make any further adjustments if required.
- Raise and lower the rod to make sure the tape does not slip.
- Check the reading.

Reflector Unit

With the sighting pole, vertical target, and tribrach in place, fasten the unit to the tripod.

Set the tripod over the point so that it is positioned within a couple of inches.

Firmly set the tripod shoes into the ground.

Use the tribrach leveling screws to get the optical plummet aligned to the ground point.

Level the reflector by adjusting the tripod legs, sliding them either higher or lower. Slight tribrach movement may be subsequently necessary, relative to the tripod head.

Orient the reflector units such that the prism axis is approximately parallel to the line of sight.

Notify the EDM operator that the unit is set.

P:HSM2

3
Electronic Surveying

WSDOT has a strategy for using computer-aided engineering, both hardware and software, to develop highway engineering designs. Survey data is collected in the field by using electronic data collectors which record height and distance measurements from prisms on the rods. The data is then electronically transferred to a computer where it is examined and edited by the surveyor, then transferred to computer programs that can be used by the highway designer.

Full utilization of the power of electronic surveying relies on the concept of a three-dimensional digital terrain model. Significant terrain features are selected, surveyed, and coded so that the computer program can digitize a representation of the existing ground. Any number of proposed highway alignments can then be investigated for cross sections, end areas for quantities of cuts and fills, and other engineering and environmental inquiry. The data can be used in computer aided drafting.

This chapter provides guidance on electronic data collection, conversion, and processing.

WSDOT uses total station equipment with electronic distance measuring capability and electronic data collectors to obtain field data, SOKKIA SDR Electronic Field Book (SDR-20 and SDR-33) to process the data and Civil Engineering Automation Library for PCs (CEAL) to create digital terrain models (DTMs), and XVIEW in the WSDOT Engineering Systems Menu to view cross sections.

Electronic surveying allows the survey crew to record and process data with a minimum of hand written field notes and very little manual data entry. The instrument person must enter the WSDOT Standard Survey Code for each point and, if desired, the first point number (subsequent numbers are electronically provided). The observation data from the prisms is transferred directly by the total station to the data collector. The field crew transfers the survey data from the data collector to a subdirectory in the hard drive of a personal computer and then processes the data in SOKKIA software (for example, by editing to assure that the points are properly coded or by adjusting a traverse). Processing of the data by the surveyor provides the most knowledgeable and therefore appropriate manipulations.

The survey crew reviews the project and selects the most appropriate method or methods of data collection for the terrain and requirements of the job. There is no right or wrong method. Data collected by photogrammetric means may be appropriate. The traditional cross-section pattern of observation points may be used.

The digital terrain modeling (DTM) method (also called selective point method) (xyz format) is the preferred method for collecting data that will be used to develop an accurate digital terrain model in CEAL. The DTM method allows the most versatility for collecting data, but proper coding is critical.

The electronic surveying system is dependent on the use of appropriate surveying procedures in the field. It is a tool which can enhance safety, efficiency, productivity, and accuracy. Data transfer is very fast and data entry errors are nearly eliminated. However, it is a very sophisticated tool and there are times when traditional (nonelectronic) surveying is more appropriate for the task.

The goal is to use an efficient process for obtaining raw data, transferring it to computers, and graphically displaying the resulting design.

Accuracy Concerns

Use of a data collector in the recording of field terrain information requires attention to standard survey practices to ensure accuracy. The lack of a paper trail (field book) can allow errors to go undetected by survey crews. The following guidelines apply regardless of the surveying method.

1. Instrument setups must be stable and solid.
2. Check into backsights/benchmarks periodically and before changing instrument setup.
3. Close and adjust project control traverse before collecting DTM or cross section data.
4. Accurately measure height of prism and instrument.
5. Double check keyboard input of all angles, coordinates, and elevations.
6. Sight on the center of the prism.
7. Ensure that accuracy standards are appropriate for the type of work being done.
8. Use WSDOT standard survey codes.
9. Make supplemental notes to accurately describe nonstandard items.
10. Calibrate equipment regularly.

Data Collector Setup

The setup of the data collector is extremely important in providing a uniform output without double corrections being applied.

Check the following items in the data collector each time a job is created.

Job Create and make a supplemental note of the name you used. (Do not use spaces or periods in the name.)

Scale Factor See Chapter 6 for instructions about when and how to make a projection from Washington State Plane coordinates to project datum coordinates. Normally we do not use the Data Collector for projections. If you are using the Data Collector for projections, use the appropriate scale factor. If not using state plane coordinates, use 1.00000000.

Point ID (SDR-33 only) Set to **Numeric (4)**.

Autopoint Number Default starts with 1000 or number(s) can be entered.

Record Elevation (SDR-33 only) Set to **YES**.

Atmos Crn (atmospheric correction) Set to **YES**. This will record the barometric pressure and temperature in your notes. Set the PPM on the instrument to **0** or a double correction will result.

C and R Crn (curvature and refraction correction) Set to **YES**. Turn on C and R in the instrument. This will not cause a double correction because the instrument only applies this correction on horizontal readings. This will assure that curvature and refraction corrections are applied whether a data collector is used or not. For the coefficient of terrestrial refraction for Washington State use **0.14**.

Sea level Crn When using state plane coordinates and a scale factor, set to **YES.** You must also use real elevations. If a scale factor is not used or you are using assumed elevations then set to **NO**.

Tolerances Set tolerances in accordance with the manufacturer's specifications for the instrument being used.

Units The following units should be used:

Angle	**Degrees**
Distance	**Meters** Also set the instrument to meters.
Pressure	**Millibar or mmHg**
Temperature	**Celsius**
Coordinates	**N-E-Elev.**

Prism Constant

2 way instruments: (Set 3BII or Set 2BII instruments newer than 1993) Set the data collector to the proper constant and it will control the instrument.

Non 2 way instruments: Set the data collector to **0** and set the instrument to the proper constant. This will allow the instrument to measure distances properly without the collector attached.

WSDOT Standard Survey Codes				
CODE	**3D**	**2D**	**3DP**	**DESCRIPTION**
AC	L			Asphalt Curb (face)
AD	L			Asphalt Ditch
APB	L			Asphalt Pavement Break
ASM	P			Aerial Surveillance Marker
BAS	L			Bridge Approach Slab
BB	L			Bank Bottom
BCUL			L	Box Culvert
BEG	L	L	L	Begins 3D or 2D line
BF			L	Barrier Face
BH	P			Bore Hole or Drill Hole
BKL	L			Breakline generic ground break
BPS	P			Back of Pavement Seat
BR		L		Bridge Rail
BRC		L		Bridge Rail Curb
BRD		P		Bridge Drain
BREJ		P		Bridge Expansion Joint
BRGR		L		Bridge Guard Rail
BRP		P		Bridge Pier
BSCK				Back Sight Check (will not be processed by the translator)
BSP	L			Bottom of Stockpile
BST	L			Edge of BST shoulder
BSWB		L		Bridge Sidewalk Back
BSWF		L		Bridge Sidewalk Face
BT	L			Bank Top
BU	L			Building (plots same direction as shots taken) Obscure area
C	P			Concrete selective
CB	P			Catch Basin (requires supp. note - inlet, outlet & type)
CBFL1		L		Catch Basin Flow Line
CBFL2		L		Catch Basin Flow Line
CBFL3		L		Catch Basin Flow Line
CBFL4		L		Catch Basin Flow Line
CBFL5		L		Catch Basin Flow Line
CBFL6		L		Catch Basin Flow Line
CBFL7		L		Catch Basin Flow Line
CBFL8		L		Catch Basin Flow Line
CF	L			Curb Face

3D L Three dimensional line with three dimensional points
3D P Three dimensional point
2D L Two dimensional line with two dimensional points
2D P Two dimensional point
3DP L Two dimensional line with three dimensional points

WSDOT Standard Survey Codes				
CODE	**3D**	**2D**	**3DP**	**DESCRIPTION**
CFTI	L			Curb Face Traffic Island
CL	L			Center Line
CP		P		Control Point
CRD	L			County Road
CTR	L			Center Line
CSP	L			Concrete Slope Protection
CUB	L			Cut Bottom
CUT	L			Cut Top
CW			L	Cross Walk
DB	L			Ditch Bottom
DGA		P		Down Guy Anchor
DI	P			Drop Inlet (requires supp. note for inlet, outlet & type)
DL	L			Driving Lane
DP	P			Drain Pipe
DW	P			Dry Well
DWE	L			Driveway Edge
DWY	L			Driveway
DY	L			Double Yellow Stripe
EB		P		Electrical Box
EGR	L			Edge of Gravel Road
EGS	L			Edge of Gravel Shoulder
EL	P			Spot Elevation
END	L	L	L	Ends 3D or 2D line
EP	L			Edge of full depth pavement (fog line)
EPS	L			Edge of Paved Shoulder
EPT	L			Edge of Pavement Taper
ERR				Error (translator will delete point)
FE			L	Fence (use param. codes for type)
FGR			L	Fence Guardrail
FH		P		Fire Hydrant
FL			L	Flow Line for pipes (use param. codes for type & size)
FS	L			Fog Stripe
FT	L			Fill Toe
G	P			Ground selective, original ground
GATE		P		Gate for Fence
GB	L			Gutter Back

3D L Three dimensional line with three dimensional points
3D P Three dimensional point
2D L Two dimensional line with two dimensional points
2D P Two dimensional point
3DP L Two dimensional line with three dimensional points

WSDOT Standard Survey Codes				
CODE	**3D**	**2D**	**3DP**	**DESCRIPTION**
GBW	L			Gabion Wall
GBWB	L			Gabion Wall Bottom
GBWT	L			Gabion Wall Top
GF	L			Gutter Face
GI	P			Grate Inlet (requires supp. note for inlet, outlet & type)
GORE	L			Gore Stripe
GP		P		Guide Post
GPS		P		Global Positioning Point (control point)
GR			L	Guardrail (requires supp. note for anchor type & height)
GRA	P			Guardrail Anchor
GUY		P		Guy Wire or Guy Pole
HW	P			Head Wall
HWL	L			High Water Line (requires supp. note with date)
IA		P		Impact Attenuator
IRSH		P		Irrigation Sprinkler Head
IRBX		P		Irrigation Box
JB	P			Junction Box (rmuires param. code for type)
LAKE	L			Lake Edge of existing water line
LE	L			Lane Edge
LP		P		Luminaire Pole
MB		P		Mail Box
MH	P			Manhole (requires param. code for type)
MON	P			Monument
O			L	Overhead Utility (use param. codes for type)
OCAB			L	Overhead Cable
OPOW			L	Overhead Power
OSIG			L	Overhead Signal
OTEL			L	Overhead Telephone
OUTIL			L	Overhead Utilities
P	P			Pavement selective
PA		P		Painted Arrow or Plastic Arrow
PBX		P		Power Box
PEDP		P		Pedestrian Signal Pole
POND	L			Pond Edge of existing water line
POST		P		Post single
PP		P		Power Pole

3D L Three dimensional line with three dimensional points
3D P Three dimensional point
2D L Two dimensional line with two dimensional points
2D P Two dimensional point
3DP L Two dimensional line with three dimensional points

WSDOT Standard Survey Codes				
CODE	**3D**	**2D**	**3DP**	**DESCRIPTION**
PROPC	P			Property Corner
RIP	L			Rip Rap Edge
RIV	L			River Edge of existing water line
ROC	P			Rock single face closest to center line
ROCS	L			Rock Outcroping
RR	L			Rail Road Center of Tracks
RRS		P		Rail Road Symbol
RRT	L			Rail Road (Top of track)
RWB	L			Retaining Wall Bottom
RWT	L			Retaining Wall Top
SN	P			Sounding
SB			L	Stop Bar
SBX		P		Signal Box
SGP		P		Signal Pole
SLAB	L			Concrete Slab
SP		P		Sign Post
SP2		P		Sign Post #2 from roadway (multipost signs)
SP3		P		Sign Post #3 from roadway (multipost signs)
SP4		P		Sign Post #4 from roadway (multipost signs)
STRE	L			Stream Edge of existing water line
STUMP	P			Stump
SW	L			Sidewalk
SWB	L			Sidewalk Back
SWF	L			Sidewalk Face
T	P			Tree - face closest to C/L (use param. codes - type & size)
TB	L			Top of Bank
TBM	P			Temporary Bench Mark
TBTH		P		Telephone Booth
TBX		P		Telephone Box
TC	L			Top of Curb
TL			L	Tree Line
TP		P		Telephone Pole
TPL	L			Topog. Limit
TSL			L	Traffic Signal Loop
TSB		P		Traffic Sianal Box
TSP	L			Top of Stockpile

3D L Three dimensional line with three dimensional points
3D P Three dimensional point
2D L Two dimension al line with two dimensional points
2D P Two dimensional point
3DP L Two dimensional line with three dimensional points

WSDOT Standard Survey Codes				
CODE	**3D**	**2D**	**3DP**	**DESCRIPTION**
U			L	Underground Utility (use param. codes for type)
UCAB			L	Underground Cable
UCB		P		Underground Cable Box
UGAS			L	Underground Gas
UOIL			L	Underground Oil
UOFC			L	Underground Optic Fiber Cable
UPOW			L	Underground Power
UTB		P		Underground Telephone Box
UTEL			L	Underground Telephone
USEW			L	Underground Sewer
USIG			L	Underground Signal
UUTIL			L	Underground Utilities
UWAT			L	Underground Utilities
VAULT		P		Underqround Utility Vault
VENT		P		Vent from underground utility
VG	P			Valve Gas
VW	P			Valve Water
WLF	P			Wet Land Flag
WLA	L			Wet Land Area
WMB		P		Water Meter Box
WW	P			Wing Wall

3D L Three dimensional line with three dimensional points
3D P Three dimensional point
2D L Two dimensional line with two dimensional points
2D P Two dimensional point
3DP L Two dimensional line with three dimensional points

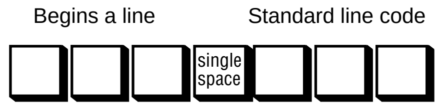
Parameter codes can be any combination of letters and numbers. They can be used after any standard code that is followed by a single space.

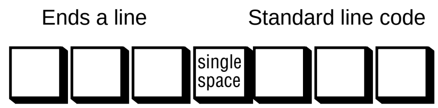
If abbreviations are used for parameters, then supplemental notes must accompany the file.

Example: **T .45 FIR** for a 0.45 meter fir tree.

Coding Examples

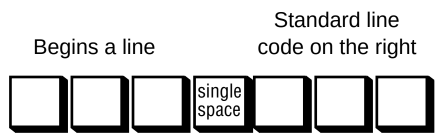
For a stringline, begin with the code that describes the beginning point, then use a single space and follow with the line code.



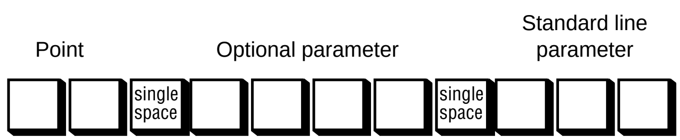


After a **BEG** code the program will string together subsequent **CTR** shots by increasing point number until an **END** code is used.

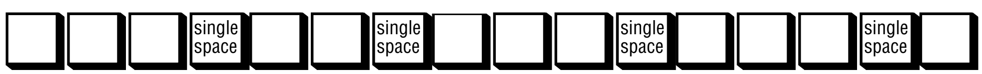
Guardrail on the right is a two dimensional line string that can cross three dimensional lines without creating discontinuity errors in CEAL. It will allow the designer easy access to the length of each section of guardrail.



For a manhole 1200 mm diameter sewer, the code is:

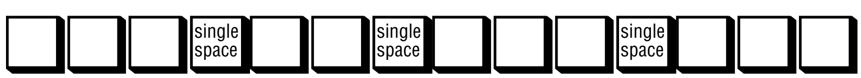


To begin a 300 mm concrete pipe that is beveled use:



This is a two dimensional line that can cross three dimensional lines such as center line, etc. This will provide the designer with the length and type of pipe and easy access to the skew angle. The grade of the pipe will be calculated from the elevations provided in SDRMAP.

The code on the other end of the pipe should read:



This ends the line string for a 300 mm concrete pipe that is not beveled on this end.

Note: A two dimensional line string can cross other two dimensional or three dimensional line strings. A three dimensional line string cannot cross other three dimensional line strings. Two dimensional points and line strings will have elevations in the data collector and in SDRMAP and CEAL and they will be used when generating a DTM file for CEAL.

Check all of these items every time a job is created.

Gathering Digital Terrain Model Data

When collecting DTM data, the most important person on the survey crew is the rod person. This person sets the pace for the crew and makes many of the decisions about what type of information the survey will provide to the design team. When selecting locations for shots, the rod person must look in all directions for terrain breaks.

Prior to beginning the survey, the crew should carefully plan which standard codes they are going to use for the breaklines.

Here is an example of a normal roadway section showing the typical standard codes.

To gather digital terrain model data:

1. Meet with design team to discuss project limits, break lines, and special features.

2. Using total station equipment, run a control traverse with approximately 450 m legs.

3. Using a bar code level or standard rod and level, run a level circuit through all control points to determine elevations.

4. Set up job in data collector and record in supplemental notes the date, the beginning and ending point numbers, job name, and the crew.

5. Set up and orient the total station.

6. Select the collimation program in the data collector.

7. Collimate on an object that is about the same distance away as the data collecting shots will be.

8. Record collimation and select the topography program in the data collector.

9. Gather crew together and discuss strategy for collection to prevent over or under coverage.

10. Begin to gather DTM data.
    (a) Check the configuration on the data collector to ensure that the auto point number is correct (do this daily). Let the data collector do the point numbering.

    (b) Paint a spot for the beginning or end of breaklines.

    (c) Collect breakline data in the direction of increasing point numbers. (Do not jump back on the same breakline.)

    (d) Use WSDOT standard survey codes and include Left or Right for plotting.

    (e) Make supplemental notes to further describe topography items such as catch basins, guardrail anchors, etc.

    (f) For accurate profiles, do not exceed 20 m spacing on pavement.

    (g) The maximum spacing between shots for CEAL is 100 m. However, for accuracy, a maximum spacing of 50 m is recommended.

    (h) Do not take shots more than 250 m away from the instrument.

    (i) The computer will draw straight lines between points, so more shots must be taken on horizontal and vertical curves.

    (j) Do not cross three dimensional breaklines.

10. Download, edit, backup, and print the data collector files daily.

Gathering Cross Section Data

After project control points have been established, the basic steps in cross section data collection are:

1. Lay out a base line (typically along center line).

2. Establish stationing (typically staked every twenty meters and at other points where the terrain significantly changes).

3. Select the collimation program in the data collector.

4. Collimate on an object that is about the same distance away as the data collecting shots will be.

5. Record collimation and select the topography program in the data collector.

6. Cross section ground breaks at each station (and other significant ground breaks) perpendicular to the base line.

When using the data collector, any time the field crew moves from one station to the next, define the first shot at a new station as a break. This is necessary for data collector program operation. A final shot must be on center line beyond the last needed cross section. It may be any distance as long as it is ahead of the last required station and on center line.

Shots may be taken in any order, and the program will sort the shots into the correct stations as long as these basic rules are followed:

1. Station break is coded each time the station changes.

2. The center line is shot at each station.

3. The center line or base line is always zero distance, left or right, in the cross section.

4. The center line is only shot once for each station.

5. In the first space of any code entered in the data collector, an alpha character is expected. At the station break, either a C or a number is acceptable. To begin the code with a number, begin the code with a blank space. For example: _20 R.

Processing Survey Data

It is the job of surveyors to provide the department with accurate data. Nobody knows the details of a survey better than the party chief and the instrument person. Therefore, the party chiefs should be responsible for transferring and editing the data.

SOKKIA

In this chapter the term SOKKIA will refer to both SDRMAP and SDRLINK. The major difference between these software programs is that SDRMAP has feature codes and plotting options that SDRLINK does not have.

WSDOT has selected SOKKIA as a standard for its linking and editing of SDR data.

An option on a menu may be selected by using the mouse, the cursor and enter keys, the numeric keys, or the **F** keys. SOKKIA will attempt to guide you through the transfer operations by highlighting an option in each menu. The **Enter** key will select the highlighted option. The **ESC** key will exit the menu back to the previous menu.

Job Creation and Selection

From the Main menu select the job creation and selection menu. If you are transferring data to SOKKIA for the first time then **[1] Start a new job**. If you are transferring to an existing job then **[2] Select an existing job**. Notice that the current job and SDR file are shown in the box at the bottom of the screen.

The Job initialization details should be filled out completely. This information is used for header information on reports generated in SOKKIA. The Job identifier has a maximum of three characters either alpha or numeric. (Most commonly the sign route of the project). The job identifier will become the extension of all files transferred to this data base. This operation is only done once for each project. Press **[F1]** when all parameters are correct

Job initialization details

Job identifier	714
Directory to store job in	C:\SDRDATA\L0123
Job name	NORTH FORK INTERCHANGE
Job description	TOPO FOR DTM
Job reference	STATE PLANE NAD 83/91
Field surveyor	MIKE OLSON
Computer operator	JEFF LANG & OLSON
Distance units	Meters
Projection	None

Press F1 when params are correct

Press ESC to exit

Transfer Data From Data Collector to SOKKIA

At the Main Menu, select option **[2] SDR Menu**

Sokkia Software V5.50 Serial number : 1000

Main Menu

[1] Job creation and selection
[2] **SDR menu**
[3] Data editing
[4] Data editing (text mode)
[5] Data comparison
[6] Utilities
[7] Configure system
[8] Exit to DOS
[9] Plotting and feature coding
[10] Import/Export menu

The current job is 714 in directory C:\SDRDATA\L0123
The current SDR file is 123T500

Then select option **[1] Receive SDR file.**

Sokkia Software V5.50 Serial number : 1000

SDR menu

[1] **Receive SDR file**
[2] Print current SDR file
[3] Edit current SDR file
[4] Transfer current SDR file to database
[5] Create an SDR file
[6] Send current SDR file
[7] Load SDR external program
[8] Select an existing SDR file
[9] Delete current SDR file
[10] Copy current SDR file
[11] Exit to main menu

The current job is 714 in directory C:\SDRDATA\L0123
The current SDR file is 123T500

The options shown in the top box of the next screen may be edited by placing the cursor on the option and striking the enter key. The options will appear in a submenu. For input device use **[1] SDR** for transferring from a data collector or **[3] Import data file** for transfers from a disk. The data transfer parameters must be set the same as the COMMS SETUP in the data collector. After you are through with the configuration, select option **[F1] Receive SDR file**. (If the data fails to transfer, try lowering the baud rate of transfer in the data collector and SOKKIA.)

After the data has transferred you have the option to edit the errors out of the file. The file editor allows you to edit the raw data from the data collector. This is where you use the supplemental notes you took in the field. If you need to edit, select **[F2] Edit current SDR.**

Receive data from SDR

Display data when ASCII receiving	No
Print messages to printer	No
Communications port	COM1
Baud rate of transfer	38400
Parity	None
Data bits	8
Input device	SDR

Press the F1 key to receive data when the SDR is connected to the computer.

[F1] Receive SDR file
[F2] Edit current SDR file
[F3] Transfer current SDR file
[F4] Continuous receive of SDR files

Press ESC to exit

You may edit an SDR file by placing the cursor over the number to be edited and typing in the new number, or by selecting **[F10]** for the options menu. To exit the file editor use the **ESC** key. This will ask you if you want to transfer SDR file to the data base. If you make any edits to an existing data collector file you must then transfer it to the data base.

SDR20 V03-05	Copyright 1985-92 by Datacom Software Research. Limited.		
	Serial no 5989	Jun-16-96 06:14	
	Angle : Degrees	Dist : Meters	Press : inch Hg
	Temp : Celsius	Coord : N-E-Elv	H. obs : Right
JOB	Job ID 123T500		
POS KI 0001	Nrth 638213.920	East 1075976.290	Elv 210.110
	Code CP		
POS KI 0002	Nrth 637789.540	East 1076014.730	Elv 206.380
	Code CP		
POS KI 0003	Nrth 637686.610	East 1076505.410	Elv 204.570
	Code CP		
POS KI 0004	Nrth 637987.330	East 1076642.490	Elv 205.830
	Code CP		
	Press F10 for Options	Press ESC to exit	

See Section 3 of the SOKKIA users guide for details on editing.

Options Menu

[1] Search for observation point
[2] Insert after current record
[3] Delete current record
[4] Undo last delete
[5] Print SDR file from current position
[6] Load main database points
[7] Abort edit session
[8] Restore raw data
[9] Global replace codes
[10] Renumber Points
[11] Replace null codes

Select **[F1] Transfer SDR file to database.** If you collected the field data in the raw format this transfer will convert and store it in the data base. The data base is the Job you created in SOKKIA.

If you have any resection or traverse shots in your data, a message will appear and ask if you want to adjust the traverse. See Section 4.3 of your users guide for details on adjusting traverses. When the transfer is complete SOKKIA will return to the transfer menu. If you do not want to process feature codes for plotting at this time, strike the **ESC** key to exit to the Main menu.

Transfer SDR file to database

Print point number re-allocations	No
Duplicate point action	Query
Duplicate point tolerance	0.000
Derive traverse from stations	TV RS SC

[F1] Transfer SDR file **123T500** to database
[F2] Select an SDR file to transfer
[F3] Edit current SDR file
[F4] Process feature codes

Press ESC to exit

Utilities and Backing up Files

Select **[F7] Utilities** from the main menu. This will allow you to back up SDR files, Jobs, and Libraries.

Sokkia Software V5.50 Serial number : 1000

Utilities

[1] Backup menu	[8] COPY SDRFILE TO A:
[2] Restore backup	[9] COPY SDRFILE TO B:
[3] Job maintenance menu	[10] DISPLAY POINT GAP
[4] Create directory	[11] PROCESS FOR CADD
[5] Remove directory	[12] PROCESS FOR CEAL
[6] System report	[13] WSPC TO PROJECT DATUM
[7] User program maintenance	[14] PROJECT DATUM TO WSPC
	[15] CEAL CHAIN TO SDR33 RD
	[16] EDITOR
	[17] CONVERT FEET\METER

Create directory
Directory **C:\SDRDATA\L0123**
ESC to cancel

Press ESC to exit

There are many ways to back up an SDR file. The SDR files are stored on the hard drive in subdirectory \SDRDATA\. They are assigned a dump number and the name you called the SDR file will not appear. The file you just transferred will be the highest dump number plus the extension of the data base you transferred it to. Example: SDR00001.714 would be the first file dumped, and it was transferred to the job with an identifier of 714.

SOKKIA allows the addition of third party programs to its utility menu. WSDOT has a program that will rename the SDR file to what you named it in the data collector and make a backup copy. The party chief should keep a backup copy with a second copy going to the office.

Data Base Editor

The data base editor can be selected from the main menu by selecting **[4] Data editing.**

This is a graphical editor that can be used to add, modify, delete, and view data base points. A detailed description on how to use the DB editor is shown in Section 7 of the SOKKIA users guide. An important thing to remember is that the points that are modified in the DB editor are not changed on the original SDR files. Therefore, back up the data base periodically using the utility menu.

Send an SDR File to the Data Collector From SOKKIA

From the SDR menu, select the SDR file that you want to send. The current SDR file name is shown in the box at the bottom of the SDR menu. The options shown in the top box of this screen may be edited by placing the cursor on the option and striking the enter key. The parameters must be the same as the COMMS SETUP in the data collector. To avoid keyboard input in the field, create an SDR file by loading main database points.

Send SDR file to SDR data collector

Display data when sending	No
Communications port	COM1
Baud rate of transfer	38400
Parity	None
Data bits	8
Output device	SDR
Load into existing job in SDR	No

Before sending the current file, connect the SDR to the computer and select the **Comms input** function from the functions menu. The SDR should be displaying **Receiving…** before you press the **F1** key to send the data.

[F1] Send current SDR file 123T500
[F2] Select SDR file to send

Press ESC to exit

Engineering Systems Menu

The engineering systems menu processes the data accumulated in the data collector, in addition to running other engineering related applications. This section will discuss the most common applications used by surveying personnel.

The engineering system menu has been developed by WSDOT for use on computers. Your data processing coordinator maintains the installation diskettes to install this system on your PC, or you may download a file from the IBM mainframe named HWY.ENGR.DATA(ENGRMENU). An option may be added to the existing menu on your computer to call up the engineering systems menu, or you can type **ENGR** at the DOS prompt C:>.

```
ENGINEERING SYSTEMS MENU
last revision - XXX  XX  XXXX

 1:  DOS SHELL
 2:  SURVEY SYSTEMS MENU
 3:  SDRLINK/SDRMAP
 4:  ALIGNMENT MANIPULATION MENU
 5:  ELEVATIONS AND VOLUMES MENU
 6:  MISC. APPLICATIONS MENU
 7:  CEAL APPLICATIONS MENU
 8:  STRUCTURES MENU
 9:  CADD APPLICATIONS MENU
10:  UTILITIES MENU
11:  USER CUSTOMIZED MENU
12:  QUIT
```

On line help is provided on each menu panel by pressing the **F1** key.

The most commonly used options by surveyors are:

2. Survey Systems Menu. This menu is used with cross section data.

3. SDRLINK/SDRMAP. This calls up a third party software (SOKKIA) used for downloading and editing SDR data collector files.

7. CEAL Applications Menu. Applications from this menu are used to convert data between survey data recorders and CEAL software.

Using SDR/DTM Survey Data to Create a CEAL Map Model

This process refers to the data collected in the field using a total station instrument with a data collector. The ground elevations and coordinates are taken selectively to produce a digital terrain model (DTM). The data is recorded in the data collector and loaded into the computer with a software package called SOKKIA. When the SOKKIA database contains the necessary points a single file is created with 08KI records. The Engineering Systems Menu creates a program (PGM) file from the 08KI records which CEAL uses to write a Map Model.

You may access SOKKIA software directly from WSDOT's Engineering Systems Menu by selecting **[3] SDRLINK/SDRMAP.**

After accessing SOKKIA, select either the current job or specify a different one. Note that the current job and its directory and the current file are displayed on the bottom of the screen.

From the main menu select **[2] SDR Menu.**

Sokkia Software V5.50 Serial number : 1000

Main Menu

[1] Job creation and selection	[9] Plotting and feature coding
[2] **SDR menu**	[10] Import/Export menu
[3] Data editing	
[4] Data editing (text mode)	
[5] Data comparison	
[6] Utilities	
[7] Configure system	
[8] Exit to DOS	

The current job is 714 in directory C:\SDRDATA\L0123
The current SDR file is 123T500

From the SDR Menu select **[5] Create an SDR file.** Enter SDR Job Identifier and SDR version (SDR 20 V03-05). The Job Identifier is the name which is placed on the first line of the SDR file. Select the appropriate SDR version and press F1 to create the file.

Sokkia Software V5.50 Serial number : 1000

SDR menu

[1] Receive SDR file
[2] Print current SDR file
[3] Edit current SDR file
[4] Transfer current SDR file to database
[5] **Create an SDR file**
[6] Send current SDR file
[7] Load SDR external program
[8] Select an existing SDR file
[9] Delete current SDR file
[10] Copy current SDR file
[11] Exit to main menu

The current job is 714 in directory C:\SDRDATA\L0123\
The current SDR file is 123T500

In the next panel, **Creating SDR file FILENAME**, move the highlight box to the field **Job ID** and press F10 to bring up options.

Creating SDR file FILENAME

SDR20	V03-05	Copyright 1985-92 by Datacom Software Research Limited.		
		Serial no 5989	16-JUN-95 06:14	
		Angle : Degrees	Dist : Meters	Press : mmHg
		Temp : Celsius	Coord : N-E-Elv	H.obs : Right
		Job ID **FILENAME**		

ESC : exit   F10 : options

From the options panel select **[6] Load Main Database Points.**

Options Menu

[1] Search for observation point
[2] Insert after current record
[3] Delete current record
[4] Undo last delete
[5] Print SDR file from current position
[6] Load main database points
[7] Abort edit session
[8] Restore raw data
[9] Global replace codes
[10] Renumber Points
[11] Replace null codes

The loading points panel will appear asking for the source of the points to be loaded, how you want the point field to be filled, and if you want codes, point numbers, descriptions, or nothing in the point field. WSDOT practice is to enter the point number in the point field and the code in the code field.

Finish the loading points panel, then press **[F1]** to continue**.**

Loading points from job 714

Source	FIELD SELECTION
Point	Point number
Code	Code

[F1] to continue

The following panel will then appear. The values displayed represent the extent of the whole database. All of the fields are editable. Only points that fit within all of the fields described are selected. When finished selecting press **[F1]** to load points.

Point range from	1	to	1719	Point class	Any		
N	637686.610	to	638386.390	chain	<Null> to	<Null> Road	<Null>
E	1075902.470	to	1076682.200	Offst	<Null> to	<Null> Code	
El	164.593	to	216.728	DesEl	<Null> to	<Null> Desc	

[F1] Load points [F2] Clear selection

Press ESC to exit

This file will reside in the default data directory specified in SOKKIA's configuration. The file will be named SDRnnnnn. Where nnnnn is a sequential number increased by 1 for each file generated. The file created will be the highest SDRnnnnn and the extension will match the database name.

Creating a .DMM file for CEAL

The .DMM file is used by CEAL to create the Digital Map Model. Creating a .DMM file uses the Engineering Systems Menu (ESM) to create a .DMM file from the previously created SDR file.

Select option **[7] CEAL APPLICATION MENU**

ENGINEERING SYSTEMS MENU
last revision - XXX XX XXXX

1: DOS SHELL
2: SURVEY SYSTEM MENU
3: SDRLINK/SDRMAP
4: ALIGNMENT MANIPULATION MENU
5: ELEVATIONS AND VOLUMES MENU
6: MISC. APPLICATIONS MENU
7: **CEAL APPLICATIONS MENU**
8: STRUCTURES MENU
9: CADD APPLICATIONS MENU
10: UTILITIES MENU
11: USER CUSTOMIZED MENU
12: QUIT

Select option **[7] SDR DTM Data to CEAL Map Model.**

CEAL APPLICATIONS MENU
last revision - XXX XX XXXX

1: Photogrammetry DGN File to CEAL Map Model - MAPCON.
2: Convert Photogrammetry DTM File to CEAL Map Model.
3: Create a Map Model Report.
4: Subgrade Elevation Listing from CEAL End Area Model.
5: CEAL SEC File to 80 Column XSEC File.
6: 80 Column XSEC File to CEAL PGM File.
7: **SDR DTM Data to CEAL Map Model**.
8: Convert a CEAL Chain to an SDR-33 Roading File-CEAL VER. 6.9.
9: Convert a CEAL Chain to an SDR-33 Roading File-CEAL VER. 7.0.
10: Quit.

A panel will appear with the cursor in position for entering the SDRnnnnn filename. This would be the file that was created in SOKKIA.

The second input field is optional. If it is left blank the Class Code File will automatically default to C:\HWYSPROD\PARM\SDRTABLE.TBL. This file contains the WSDOT standard survey codes.

The third input field is the CEAL File Names. This is also optional. If left blank the program will use the input file name and build them with the extensions of .PGM, .DMM, and .RPT.

The beginning feature string number defaults to F001 but the user may change it to start with any alpha character except **C**.

Process 08KI data defaults to yes. This needs to be yes since the file was created is 08KI records.

Parameters as notes: This option will take any parameter codes used by the survey crew to further define a point. It will be sent to the Map Model as a note record with the associated point. Toggle to **NO** if using a version of CEAL earlier than 6.71.

When satisfied with the options selected on the screen, click the mouse on **OK** or press the enter key.

```
---2.0-- SDR2 Digital Terrain Model To CEAL MAP Model -----
```

```
 SDR2 Input File ==> C:\SDRDATA\SDRnnnnn.dat         REQ
   (F1 to select)

 Class Code File ==>                                 OPT
   (F2 to select)    (default extension = .TBL)

 CEAL File Name  ==> C:\XCEAL\DATA\                  OPT
(No Extension - Used To Build .PGM and Map Model Names)

 Beginning Feature String Number ==> F001            OPT

 Process 08KI Data   :  [Y]    [N]

 Parameters as Notes :  [Y]    [N]


                  +-------+          +-----------+
                  |  OK   |          |  CANCEL   |
                  +-------+          +-----------+
```

This file will be displayed if you did not specify a table when you execute the program. You may select by class code the items that you want to process into CEAL. The default is yes to all codes.

```
- 2.0 - Convert SDR2 Data To CEAL Map Model ----F1 = HELP -
```

```
[Y]  [N]  :  ROADWAY FEATURES (LANE EDGE)
[Y]  [N]  :  CTR      - Center Line
[Y]  [N]  :  DL       - Driving Lane
[Y]  [N]  :  DLL      - Driving Lane Left
[Y]  [N]  :  DLR      - Driving Lane Right
[Y]  [N]  :  EP       - Edge of Full Depth Pavement
[Y]  [N]  :  EPL      - Edge of Full Depth Pavement Left
[Y]  [N]  :  EPR      - Edge of Full Depth Pavement Right
[Y]  [N]  :  FS       - Fog Stripe
[Y]  [N]  :  FSL      - Fog Stripe Left
[Y]  [N]  :  FSR      - Fog Stripe Right
[Y]  [N]  :  LE       - Lane Edge
[Y]  [N]  :  LEL      - Lane Edge Left
[Y]  [N]  :  LER      - Lane Edge Right

 +------+     +----------+     +-------+     +--------+
 | NEXT |     | PREVIOUS |     | ENTER |     | CANCEL |
 +------+     +----------+     +-------+     +--------+
```

You have the option of saving a table that has been modified. An example of this would be to save a table that only translates 3D data or 2D data.

```
             *****  SAVE TABLE OPTION  *****

    Customized tables can be saved for use in later
    runs of the SDR Data to CEAL Map Model
    translator.  Select YES to save the table or NO
    to skip the save and continue processing.

                    +-----+     +----+
                    | Yes |     | No |
                    +-----+     +----+
```

If a code is used that is not a standard survey code, the program will display the following option.

```
              *****  INVALID CLASS CODE  *****

    A class code of RIVER  was found in the SDR data
    file.  This class code is invalid and does not
    exist in the class code table.  Select YES to add
    this class code to the table for this run.
    Selecting NO will cause all records with this
    class code to be skipped.

                     +-----+     +----+
                     | Yes |     | No |
                     +-----+     +----+
```

If **Yes** is selected, the following panel will be displayed.

You have the option to change the old code to a standard code, this will save having to modify the schedule file in CEAL.

```
 ------------- Temporary Class Code Type ----------------
```

```
       Old Class Code  ==> RIVER

       New Class Code  ==> RIV                  OPT

       Class Code Type ==> 3DF                  REQ

Valid String Types:  3DF  OBS  APP  BLD  2DF  3DP
Valid Point Types:   GP   GS   FP   GX   RP

            +----+              +--------+
            | OK |              | CANCEL |
            +----+              +--------+
```

You must tell the program what class code type the new code is. The following is a list of valid class code types.

String Types When used with BEG and END codes.

*	3DF	3D Feature string
	OBS	3D Obscure area (closed shape)
	APP	3D Approximate area (closed shape)
	BLD	3D Building (closed shape)
*	2DF	2D Feature string
	3DP	2D Feature string with 3D points

Point Types

*	GP	3D Feature point
	GS	3D Special feature point (local high, low, saddle)
*	FP	2D Feature point
	GX	2D Subsurface feature point
	RP	2D Rock layer subsurface feature point

* most commonly used

When processing is complete, a .DMM, a .PGM, and a report file will be created. The *.PGM file will contain two points, the minimum and maximum coordinate values (lower left and upper right corners) for indexing in CEAL. The *.DMM is the Map Model file for CEAL, and the *.RPT is the report file.

```
           *****  SUCCESSFUL COMPLETION  *****

     Program SDR2CMAP for converting SDR2 Digital Terrain
     Model data to CEAL MAP Model Format has completed
     successfully.

     Map Model File = C:\XCEAL\DATA\sdrnnnnn.dmm
     CEAL .PGM File  = C:\XCEAL\DATA\sdrnnnnn.pgm
     Summary File   = C:\SDRDATA\sdrnnnnn.rpt

                  Processing is now complete.
```

The report file resides in the same directory as the input file and has an extension of .RPT. The report file should be printed. It contains the errors, chains created, and codes added or ignored.

```
Found Error Code 1629
Invalid Class Code - WS Elements With This Class Code Being
Skipped.
**************************************************************
Temporary Class Code <RIVER> Added To Job As class code <RIV>
**************************************************************
Found Feature Code RWB      Assigned Chain F002
Found Feature Code SWB      Assigned Chain F003
Found Feature Code SWB      Assigned Chain F004
Found Feature Code LE       Assigned Chain F005
```

Viewing the Map Model Using CEAL Software

On the computer execute CEAL by typing **XCEAL** or **CEAL**. At the dot prompt, enter the command **GEDIT MAP fname**. Gedit map can view a Map Model and construct contours by following the menu options.

Running the .DMM file Using CEAL Software

1. After calling up CEAL, enter the command **PROGram Recall** with the .PGM file created on your computer (2.132 in the CLM Utilities manual). This will recall the minimum and maximum coordinates of the file. Do not recall other coordinate data or the index command will establish an erroneous scale.

2. To view the points in CEAL, use a **SHEET** command (3.220) followed by an **INDEX TO FIT** command (3.242).

3. Issue a **MODel FILE fname** command followed by a **PLot MAP** command to plot the Map Model.

4. To create contour and DTM models, use the construct contour and DTM model commands in 4.670 of the CLM TOPO/ROADS manual.

4

Horizontal Control

Research, Reconnaissance, and Project Layout

Upon receiving a request for a survey, research is the first step performed. Obtain existing maps of the area. These would include DNR 1:100 000 ownership, USGS 1:24 000 topography, statewide sign route air photography, and county maps to plot control schematics, locate records detailing descriptions, and coordinates from Geographic Services Survey Section or region headquarters. These records should consist of GPS, triangulation or traverse stations with NAD83/91 horizontal control and vertical control marks with NGVD29 or NAVD88 elevations.

Plot all marks useful to the project on a map that has a scale suitable for your particular project.

Instructions for the reconnaissance include the area to be covered, accuracy specified for the project, desirable spacing of control points, and connections to existing surveys including new GPS stations. As a general rule, reconnaissance personnel are responsible for satisfying all requirements for strength of figure, line of sight, and other technical specifications. Also gain permission to enter private land at this time.

Armed with a folder full of maps and descriptions, and an understanding of what it is the survey is to accomplish, a trip to the field is in order. Reconnaissance can be a time-consuming ordeal in locating the primary control marks as descriptions are often outdated. Laying out the survey to fit the needs of the project is also time consuming. This is why the recon should be done well in advance of any survey observations.

Primary, Secondary, and Topography Surveys

Most projects begin with primary control (see *Design Manual* Chapter 1450). Concrete geodetic monuments with geographic positions, latitude and longitude converted to state plane coordinates, should be in place before any other surveying commences. Primary control consist of GPS, triangulation, or traverse stations with second order class II or better accuracy. They must also have NAD83/91 values which supersedes coordinates from other systems. Legislative law dictates we use the new adjustment, and the only way to do it is with a physical tie in the field.

One efficient way to bring primary control onto a project is by the use of GPS. The exact number of GPS stations needed depends on the nature of the project. If GPS methods are not used, then use conventional traverse methods following second order class II procedures.

For a 10-kilometer project, one would expect to put in at least six monuments. There should be two intervisible stations at the beginning, two near the middle, and two near each end of project. This gives a starting azimuth, an azimuth check half way through, and a closing azimuth to finish up. Not having to traverse more than 3 to 5 kilometers without the benefit of an azimuth check, should enable a crew to use third order procedures with excellent results.

Primary control survey equipment must at least meet second order class II specifications. Once the instruction requirements are met, procedure must be rigidly followed.

For instrument requirements, minimum number of sets and rejection limits from the mean see Figure 4-1.

Classification	First Order	Second Order				Third Order	Fourth Order
		Class I		Class II			
Relative accuracy between directly connected adjacent points.[5] (at least)	1 part in 100,000	1 part in 50,000		1 part in 20,000		1 part in 10,000	1 part in 2,000
Recommended uses	Primary National Network metropolitan area surveys, scientific studies.	Area control which strengthens the National Network. Subsidiary metropolitan control.		Area control which contributes to but is supplemental to the National Network.		General control surveys referenced to the National Network. Local Control surveys.	Cross-sections. Minor Structures. Topography Preliminary lines.
Recommended spacing of principal stations	Network stations 10-15 km. Other surveys seldom less than 3 km.	Principal stations seldom less than 4 km except in metropolitan area surveys where the limitation is 0.3 km.		Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.		Seldom less than 0.1 km in metropolitan surveys of this order. As required for other surveys.	As required.
Horizontal directions or angles note #9							
Instrument[2]	0.2″	0.2″	1.0″	0.2″	1.0″	1.0″ - 3.0″	3.0″ - 5.0″
Minimum Number of sets[8]	16	8	12	6	8	4	2
Rejection limit from mean	4″	4″	5″	5″	5″	5″	5″
Length Measurements							
Standard error[1]	1 part in 600,000	1 part in 300,000		1 part in 120,000		1 part in 60,000	1 part in 5,000
Minimum number of readings	10	10		10		5	4
Reciprocal vertical angle observations[4]							
Number of and spread between observations	3 D/R - 10″	3 D/R - 10″		2 D/R - 10″		2 D/R - 10″	3 D/R - 30″
Number of stations between known elevations	4 - 6	6 - 8		8 - 10		10 - 15	20 - 30
Astro azimuths							
Number of courses between azimuth checks[6]	5 - 6	10 - 12		15 - 20		20 - 25	40
Number of observations/night	16	16		12		8	2
Number of nights	2	2		2		1	1
Standard error[3]	0.45″	0.45″		1.5″		3.0″	15″
Azimuth closure at[7] or azimuth checkpoint not to exceed	1.0″ per stations 2.0″ $\sqrt{N}$	1.5″ per station or 3.0″ $\sqrt{N}$. Metropolitan area surveys seldom to exceed 2.0″ per station or 3.0″ $\sqrt{N}$		2.0″ per station or 6.0″ $\sqrt{N}$. Metropolitan area surveys seldom to exceed 4.0″ per station or 8.0″ $\sqrt{N}$		3.0″ per station or 10″ $\sqrt{N}$. Metropolitan area surveys seldom to exceed 6.0″ per station or 15″ $\sqrt{N}$	15″

(Notes for this figure are on the next page.)

Classification for Horizontal Control Surveys
Figure 4-1

Notes for Figure 4-1

1. The standard error is calculated by:

$$\sigma_m = \sqrt{\Sigma v^2 / n(n-1)}$$

where σ_m is the standard error of the mean, "v" is a residual (that is, the difference between a measured length and the mean of all measured lengths of a line), and "n" is the number of measurements.

The term "standard error" used here is computed under the assumption that all errors are strictly random in nature. The true or actual error is a quantity that cannot be obtained exactly. It is the difference between the true value and the measured value. By correcting each measurement for every known source of systematic error, however, one may approach the true error. It is mandatory for any practitioner using these tables to reduce to a minimum the effect of all systematic and constant errors so that real accuracy may be obtained. (See page 267 of Coast and Geodetic Survey Special Publication No. 247, *Manual of Geodetic Triangulation*," Revised edition, 1959, for definition of "actual error.")

2. The figures for "Instrument" describe the theodolite recommended in terms of the smallest reading of the horizontal circle (least count).

3. The standard error for astronomic azimuths is computed with all observations considered equal in weight (with 75 percent of the total number of observations required on a single night) after application of a 5-second rejection limit from the mean for the first and second order observations.

4. See FGCC "Detailed Specifications" on "Elevation of Horizontal Control Points" for further details. These elevations are intended to suffice for compensations, adjustments, and broad mapping and control projects, not necessarily for vertical network elevations.

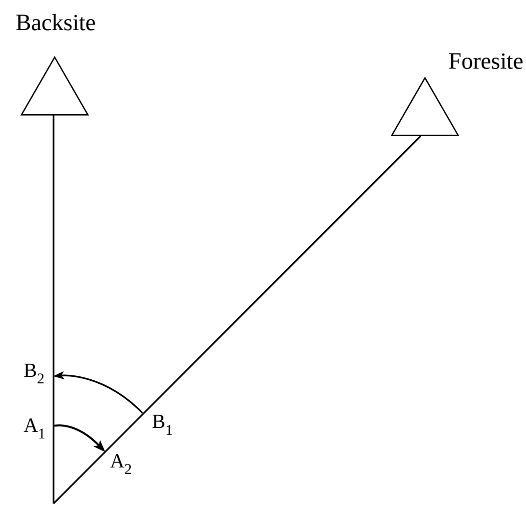
5. Unless the survey is in the form of a loop closing on itself, the position closures would depend largely on the constraints or established control in the adjustment. The extent of constraints and the actual relationship of the surveys can be obtained through either a review of computations, or a minimally constrained adjustment of all work involved. The proportional accuracy or closure (for example, $\frac{1}{100,000}$) can be obtained by computing the difference between the computed value and the fixed value and dividing this quantity by the length of the loop connecting the two points.

6. The number of azimuth courses for first order traverses are between Laplace azimuths. For other survey accuracies, the number of courses may be between Laplace azimuths and/or adjusted azimuths.

7. The expressions for closing errors in traverses are given in two forms. The expression containing the square root is designed for longer lines where higher proportional accuracy is required. The formula that gives the smallest permissible closure should be used. N is the number of stations in the traverse.

8. A set is two measurements, the telescope direct and reversed, of the horizontal direction from the initial station to each of the other stations.

Refer to the following for turning a set:



Procedure for Turning a Set

A_1 = Initial pointing: site on backsite with zero or plate setting set into instrument.

A_2 = Turn to foresite and read direct angle.

B_1 = Invert scope, reverse instrument (turn 180°), resight on foresite, read angle (the difference between A_2 and B_1 is called "split").

B_2 = Turn to backsite and read reverse angle.

Find the mean of A_1 and B_2.

Find the mean of B_1 and A_2.

Subtract the backsite reading from the foresite reading to determine angle.

This procedure may be used for multiple pointings.

9. The number of sets and the rejection limits may be modified to fit the circumstances such as for traverses having very short courses.

Exactly how much horizontal control will be needed on a project must be determined in the field. Most crews run into trouble traversing a winding road with portions having long tangents. They will take long shots down the tangents followed by a series of short, choppy shots to get around the curves. This imbalance results in closure problems. A good least squares adjustment, showing error ellipses and PPM for each line, will verify this. Many times, the short segments will be well under one part in 10 000, violating state law regarding survey standards. There are two ways to correct this. Balance traverse segments by breaking up the long tangents into distances not much further than the short shots around the curves, or put in two control points to close off the curving section, thus isolating the short shots, and running ahead from there with longer spacing. Other than mixing up datums and coordinates, this is probably the greatest source of error in traverse misclosure.

Once the primary control is established, whether it be GPS or conventional, a secondary traverse must be run on which to base the topography work. Third order specifications will be used to traverse between primary stations, with topographic work (fourth order) based upon this secondary traverse.

Monuments for primary surveys usually consist of a brass disk set into a concrete monument or structure. The work should be accomplished by using GPS equipment or if conventional, by instrumentation having a least count of 1 second or less, and an EDM with a standard error of 1:120 000 or greater.

Monumentation for secondary surveys (third order traverse) would consist of iron pipes or rebar with caps set to a depth in which to be stable. Instrumentation here would consist of GPS and conventional equipment. It is possible to use an instrument with a least count greater than 1 second.

Fourth order would consist of wood hubs, pk nails, scribe marks, etc., using equipment designed for topographic work.

Station designations now are as follows:

Primary Surveys

GPS = GP34005-6
Terrestrial = HC 34005-7

GP — Designates work done by global positioning.

34 — Stands for county number by alphabet.

6 — Being the next consecutive number by county.

HC — Designates "horizontal control" which is different from the old "TS" (topographic survey) which predates NAD 83/91.

Secondary Surveys

GPS = IS3408
Terrestrial = ?

"IS" designates "instrument station." Terrestrial or conventional secondary surveys currently have no set designation.

Secondary surveys performed by GPS generally have the same results as primary surveys with the exception that less distance between stations will bring down the PPM and usually increase error ellipses. The real difference is in the type of monumentation used. A pipe or rebar will not hold its position as well as a concrete monument, but is much more cost effective and time efficient when many are needed.

Example

You are handed a project which basically involves making a topographic map of a 6-mile stretch of highway. The first two miles are very winding with the rest clear. Primary control would most likely consist of two GPS stations near the beginning, two at the end of the winding section, with another two or three GPS stations, possibly the last three intervisible, throughout the other four miles. Then, a secondary traverse (third order) is run around the winding section and between the other GPS stations. Only then should topographic work (fourth order) be considered. The secondary traverse and the topographic work may be run concurrently, as long as you keep procedures straight. By looking at specifications, we mean third order to be four sets of angles, direct and reverse with at least five distances. Topographic work (fourth order) being one or two angle-distance data collection. Secondary traverse will be tripod to tripod, not range poles. Using good procedures not only makes your work look good, but is required.

Project

Primary Survey

GPS (Order C)
* Static techniques
* Fast static techniques
* 5 mm, 0.5 ppm
* Concrete monuments/brass caps

Conventional (2nd Order or Better)
* 8 sets direct and reverse angles
* 10 distances
* 15-20 courses between azimuth checks
* 1 second instrument or better
* Concrete monuments/brass caps

Second Survey

to Densify Primary — If more than 3 km between primary stations

* Third order procedures
* Iron pipe or rebar with aluminum caps
* Photogrammetry
* Right of way
* L-line and P-Line
* Major structures (construction)
* Traverse for topography or as-builts
* 1" to 3" instruments
* 4 sets of direct and reverse angles
* 20 to 25 courses between azimuth checks

If less than 3 km between primary stations

Topography Survey
* Fourth order
* DTM
* Advance Plans
* Minor structures (construction)

* As-builts
* HUB and tack
* 3" to 5" instruments

Project Survey Procedure
Figure 4-2

Horizontal Directions

Quality requirements for instruments used in horizontal directions must meet defined specifications. First order instruments have a direct reading of less than one second. Second order instruments can have a direct reading up to one second. Third order instruments can have a direct reading up to three seconds. A high quality surveyor's transit or a repeating theodolite is acceptable for third order surveys. Neither instrument is recommended, however, because of the extra effort required to obtain specified accuracies.

The entries under "Horizontal directions or angles" in Figure 4-1 specify the number of sets to be observed and the accuracy requirements of instruments to be used for each order and class of survey. These criteria, derived from experience and statistical evaluations, are based on the accuracy to be obtained, the instrument used, and to some extent the length of the lines involved. The requirements specify that the micrometers must be brought into coincidence twice (or two readings with an electronic) and both readings recorded. The mean is then used to determine the direction. The rejection limit from the mean must not exceed 4 seconds for first order instruments.

Degrees and minutes must be read and recorded for each observation. The practice of reading and recording only the seconds for specific portions is not acceptable. These observing procedures and tolerances also apply to measuring vertical angles.

Observations should be made using procedures that minimize collimation, circle, and micrometer errors.

When high precision is required, special care must be exercised in observing horizontal angles over inclined lines. The level of the instrument must be carefully maintained for lines that incline as much as 2 degrees. For lines inclined more than 5 degrees, it is desirable to record striding level or plate level readings using the procedures established for astronomic observations and then correct for the directions involved.

It is important for the observer to center the instrument carefully. Protect the instrument from the heating effect of the sun and from vibrations caused by wind. Center and point the instrument with utmost care to eliminate phase error in the target. Phase error is caused by light illuminating the target in an oblique manner which influences the observer to miscenter. If the edges of the target can be determined, use double-vertical cross hairs rather than the single. If possible, it is advisable to wait for better observing conditions. Schedule observations taking into account those situations in which adverse horizontal refraction is likely to be present. In these cases, make the observations during a period when atmospheric conditions will likely permit optimum results.

Electronic Distance Measurements

Distances measured with electronic distance measuring (EDM) equipment are subject to errors arising from the instrumental components, calibration of the equipment, inaccuracies in the meteorological data, elevation discrepancies, and the centering of the instruments or reflectors. These factors, as well as the operating procedures, are considered when deriving specifications for length measurements for a particular standard.

Instrumental errors are usually described in the manufacturer's specifications as a number of millimeters or centimeters, plus or minus a number of parts per million. Various tests have indicated that these statements of accuracy are reasonably valid. However, it must be emphasized that these are average values of results obtained under average conditions, often at a single location, and may not be completely representative of actual field measurements. Nevertheless, these specifications dictate the minimum line length that should be measured with a particular instrument. When measuring long distances, errors are introduced by the inaccuracies of meteorological observations. For short-range measurements, centering the instruments or reflectors and determining elevations for the reduction of slope distances to the horizontal are especially critical.

The minimum distance that should be measured with this equipment depends on four major factors: (1) accuracy required, (2) standard error of a single observation as stated by the manufacturer, (3) number of complete measurements, and (4) observation procedures.

Assuming that precise observing procedures are followed and centering errors are negligible (which may or may not be the case, because extreme care is required to achieve centering accuracies of 1 mm or less), the following formula provides a guide for measuring the minimum required distance:

$$\text{Minimum distance} \cong \frac{\sigma}{x}$$

where "σ" is the standard error expressed in millimeters and "x" is a factor that varies with the accuracy expected from the order and class. The values of x are:

first-order surveys	0.005
second order, class I surveys	0.010
second-order, class II surveys	0.025
third-order	0.050

When complete multiple measurements are observed, the acceptable minimum distance can be determined by dividing the value derived in the formula above by $\sqrt{N}$ where N is the number of complete measurements. For example, for second-order, class II accuracy, with σ = 5 mm, the minimum distance is 5/0.025 = 200 m. If two complete measurements are made (N = 2), the minimum distance shall not be less than $200/\sqrt{2}$ = 140 m. Some manufacturers state a minimum distance that can be measured. At no time should the stated minimum distance be violated.

Three types of EDM are now in use. They are electro-optical devices that employ visible light, short-range equipment that uses infrared as the carrier wave, and units that rely on microwaves. Microwave equipment and electro-optical instruments, especially those equipped with a laser light source, can measure distances of 100 km or more. Instruments with infrared sources have a limited range, generally 10 km or less.

Some infrared instruments have direct readouts, others indirect. The indirect readout instruments require a series of operations to obtain the slope distance. Their stated accuracies range from 2 to 10 mm, and include 1 ppm times the length (or more) in their statement of accuracy.

A complete measurement using direct-readout equipment consists of the minimum number of readings for length measurements shown in Figure 4-1.

Tolerance between readings should not be greater than 5 mm.

These specifications and permissible tolerances apply generally to measurements obtained with instruments of limited range (3 to 4 km or less). Some instruments are equipped with a built-in computer to correct the measurements for atmospheric conditions. Check the corrections being applied to the instrument periodically to detect any malfunction in the computer. Experience shows that more accurate and reliable results are obtained by measuring the appropriate atmospheric conditions and applying the usual formula reductions.

Excluding the effect of humidity, errors of 1°C in air temperature and 3 mm in barometric pressure (about 30 m on an altimeter) produce inaccuracies of 1 ppm for most electronic distance measuring instruments now in use. The FGCC highly recommends that meteorological data be obtained at both ends of the line for distances measured in connection with all permanently monumented horizontal control surveys. Experience indicates that when the measurements are made at tripod height, temperatures taken at about 10 m above ground level generally are more representative of conditions along the ray path. However, when the ray path barely clears the ground, temperatures obtained at instrument height are usually best. Distance measurements over such terrain, particularly in excess of 5 km, should be avoided; it is questionable whether the temperatures are representative of the actual conditions even if they were obtained at instrument height or 10 m above the ground. Always shade meteorological equipment from the direct rays of the sun. When highest accuracy is required, instruments should also be protected from the elements (for example, by using umbrellas or observing tents).

For lines exceeding 20 km, where the highest accuracy is required, it is recommended that simultaneous observations be made of the reciprocal vertical angles prior to and after the length observations measurements. These observations are used to correct the refractive index observed at each end of the line which, when meaned, produce a more representative value of the atmospheric conditions along the ray path. Adverse atmospheric conditions, such as temperature inversion, may cause this correction to be significant for elevation differences as small as 100 m. Therefore, in general, this correction should be applied whenever distances are measured for first- and second-order, class I surveys where elevation differences exceed 500 m. To determine this correction, which is actually a combination of two corrections (the second-velocity correction and the index-rate correction), requires that the elevation differences be known, as a general rule, to a few tenths of a meter.

Service all electronic distance measuring devices regularly and check frequently over lines of known distances. Calibrate instruments at least annually, preferably every 6 months. Frequency checks are recommended every 3 to

4 months. Equipment must be handled with care and protected from the elements at all times. (See WAC 332-130-100, Equipment and Procedures.)

Elevations of Horizontal Control Points

Whenever distance measurements are made in the course of performing control surveys to provide scale or, as is the case in trilateration and traverse, a fundamental element in the positioning of points by these methods, it is necessary to reduce the measurements to the horizontal and to the mean sea level surface (geoid). For highest accuracy, it is often advisable to reduce distance measurement to the reference ellipsoid. For first- and second-order, class I surveys, the use of elevations is the preferred method. Although accurate elevations are desirable, the accuracy of the differences of elevation (as opposed to absolute elevations) is of greater concern, especially where these differences are responsible for slope corrections in excess of 0.1 m. The reductions are computed using the elevations of the points at which the measurements were made, plus the accurately measured heights above the stations of the instruments or reflectors.

It is preferable to determine by spirit or compensator leveling methods, either directly (that is, the stations also serve as bench marks in a leveling line) or indirectly by connections to the bench marks. Because some control stations are located in outlying areas, direct ties may not be possible. Most of the elevations for these points are determined by trigonometric leveling using zenith distance observations — a form of vertical angles — in the computations.

Although the use of elevations to reduce measurements is highly recommended for second-order, class II surveys and surveys of lower accuracy, particularly those involving trilateration, vertical angles may be used for the reduction to the horizontal. Caution must be exercised when the cosine of the vertical angle is used to reduce slope measurements to the horizontal. If the observations are made only at one end of the line, the correction for curvature and refraction should be applied as a general practice, even if it does not significantly affect the cosine. This correction is not necessary when observations are made at both ends of the line and a mean vertical angle is used. In addition, any appreciable difference (and this can be as little as 0.3 m) in the heights at which the vertical angles and the length measurements were secured must be taken into account as another correction to the observed vertical angle. The best procedure is to compute the elevations of the marks at the two ends of the line by the established zenith distance (vertical angle) methods (reciprocal or otherwise), taking into account instrument and target height. The resulting elevations are then used to convert the slope lengths to the horizontal and/or to mean sea level distances by standard procedures.

Geographic Services can supply an N.G.S. program to reduce slope distance to geodetic (ellipsoid) distances using barometer, temperature, elevations, and height of instrument.

The entries under “Reciprocal vertical angle observations” in Figure 4-1 provide specifications for number and spread between observations. Also given, is a limitation on the number of stations over which trigonometric leveling can be carried before a check on the elevation is required. This check may be made against previously determined trigonometric elevations derived from observations obtained using similar specifications as required for the particular survey or from elevations from spirit or compensator leveling.

Although procedures for traverse are essentially the same, special emphasis must be placed on observations over short or steep lines, which are common to these surveys. Because redundant observations are lacking, extra care must be exercised.

A single set consists of one direct and one reverse pointing. If the permissible spread between the observations in a set is exceeded, the observations must be repeated until a satisfactory set is obtained. The heights of the instrument and target reflectors above the marks must be accurately measured.

Exercise special care where the difference in elevation divided by the distance exceeds 0.03 (10 m ÷ 300 m = 0.03). When this ratio approaches 0.1 and the distances are less than 300 m, consider leveling between the points.

Refraction is a major consideration in obtaining accurate vertical angle observations; observations should be made when atmospheric conditions that cause vertical refraction are most stable. In most areas the best time period for vertical angle observation is between 10 a.m. and 4 p.m. Reciprocal vertical angle observations should be made as nearly simultaneously as possible during the 10 a.m. to 4 p.m. period.

Connections to Existing Surveys

Where somewhat parallel traverses or surveys established by other methods approach each other, they should be connected if practical. The 20 percent rule is applied in this instance. This rule, formulated by the U.S. Coast Guard and Geodetic Survey, states that whenever the distance between two unconnected points is 20 percent (or less) of the sum of the distances between directly connected stations, a connection between the unconnected points should be made if possible and practical. See Figure 4-3.

Many problems with the present national network and with numerous local systems resulted from omission of connections to established control at the time nets were observed and adjusted, even though the connections were included in subsequent surveys. With the advent of Geographic Information Systems (GIS) these connections become increasingly important.

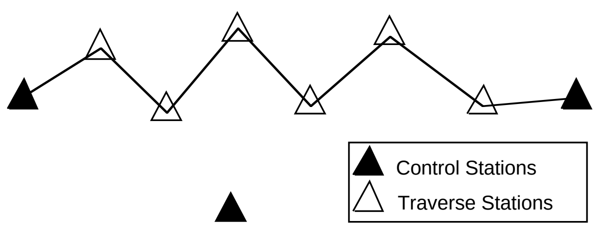


Figure 4-3

An integral part of the recovery of a station involves the reobservation and remeasurement of the directions and distances to the reference marks. The directions to the azimuth mark and to previously observed landmarks, such as water tanks and church spires, as well as other stations, should be included in the reobservation program. When significant differences are found, verify the newly obtained values in the field and notify Geographic Services Survey Section. In general, investigate observations failing to check other stations by more than two seconds. Note directions and distances to reference marks failing to check by more than three seconds and one centimeter, respectively. Azimuth marks and intersection station should be within ten seconds.

If cost factors make this recommendation prohibitive, a minimum requirement would be the verification of the distances and directions to the reference marks.

It is much better to orient a survey through azimuths to other distant stations, which were previously observed as part of the existing survey, than to use a new nearby azimuth mark.

The same holds true when working with GPS stations even though most do not have reference marks. Make a distance check between GPS monuments and reduce to state plane. Report discrepancies more than a centimeter.

New surveys normally begin and terminate at control points with an accuracy equal to or higher than that required for the new work. This does not preclude connecting higher order surveys to lower order surveys in special class.

When a project has both primary and secondary monumentation, use the primary control whenever possible.

Length, Azimuth, and Position Closures

In theory, these closures are calculated after the geometric conditions for triangulation and trilateration, and the azimuth closure for traverse have been accounted for in a minimum constrained adjustment. However, if the computations are performed in the field or if computers and adjustment programs are not available, the following practices provide results which, as a rule, are good approximations of those obtained from minimum constrained adjustments. For triangulation surveys, the triangle closures are distributed equally on each angle of the respective triangles and the sides are calculated. The length closure is obtained by computing through the strongest chain or chains to the lines of known length.

Geographic positions are then computed over the same chains to determine the azimuth and position closures. For trilateration, the measured terminal sides furnish the length checks. Azimuth and position closures are secured by computing through the strongest chain or chains of triangles using the completed angles and measured lengths.

In traverse, the azimuth closures are calculated by preliminary computations using either an initial grid azimuth or geographic positions. On first consideration, the use of an initial grid azimuth seems the simplest procedure; however, the observed angles should be corrected for the arc-chord (t-T or second term) corrections, which involves additional effort even when using approximate formulas. The distribution of the azimuth closures in traverses, as gained from long experience, is best handled by applying the first closure in equal amounts to the traverse angles, taking the route that contains the fewest number of angles between fixed azimuths. The second and succeeding closures are distributed in the same fashion using the corrected azimuths, where applicable, from previous computations. When a survey includes a combination of methods, a careful review and judicious utilization of the procedures described will generally provide an acceptable basis for judging whether the closures meet the criteria.

Accuracy and Precision

Two terms common to surveying are accuracy and precision. They are commonly used without a true distinction between them. The definitions by the National Geodetic Survey are as follows:

Accuracy — degree of conformity with a standard.

Precision — degree of refinement in the performance of an operation or in the statement of a result.

Accuracy relates to the quality of a result, and is distinguished from *precision* which relates to the quality of the operation by which the result is obtained.

Accuracy is a function of precise methods, precise instruments, and most of all, good planning. Precise procedures and methods must be used. Precise instruments are not a must, but the time spent at a station may be important and you will have to turn a lot more angles to prove your accuracy with less precise instruments. By good planning and a reconnaissance trip, you may save many man-hours later.

Precision may be defined as the degree of perfection used (the proper instrumentation, procedures, and observations). Accuracy is the degree of perfection obtained. Actual results must be used to compute accuracy. When the results do not compare favorably with the estimated results, it should be assumed that errors exist which should be corrected.

Errors

Errors are of three general types; blunders, systematic, and accidental. A blunder is a mistake in determination of a value. Eliminating blunders is one of the most important elements in surveying.

Three basic rules for eliminating blunders are:

1. Every value recorded in the field must be checked by some other field observations.

2. When this check indicates that there are no blunders, the field records must never be changed, a black pen is the preferable instrument used for manual recording in the field. If a mistake is made, draw a line through it and make the correction above it.

3. An over-all check must be applied to every control survey. As many checks as possible should be programmed in the planning of the project.

Systematic errors are errors that, under the same conditions, will always remain the same in size and sign. These errors can only be located by recognizing conditions that create them; they are therefore very dangerous. Make every effort to recognize any conditions that cause them and to take the necessary steps to neutralize them. Most surveying equipment, when used with the proper procedures, will automatically cancel most of these errors. Evaluate the errors that cannot be eliminated and determine the conditions that cause them.

Accidental errors represent the limit of precision in the determination of a true value. They obey the laws of probability. Errors of a properly conducted survey can be treated as accidental.

Error Sources

Observations
Targets
Techniques
Procedures
Measurements
Atmospheric Corrections
Geometry (design)
Adjustments (office)

Error Causes

Personal Limitations
Resolution of Eyes
Ability to Interpolate

Manual Operations
Manipulation Errors
Incorrect Procedure
Transposition of Figures
Human Mathematical Errors
Fraud

Instrument Limitations

Stability of Construction
Stability of Support
Telescope Resolution
Least Count
Targetry

Nature

Temperature
Humidity
Wind
Refraction
Gravity
Magnetic Field

Other Sources of Error

The specifications stated below include requirements for instrument accuracy, the number of necessary observations, and the required computational closures which must be satisfied to give reasonable assurance that the stated standard is met. Some of the other sections mention additional precautions. If these precautions are not taken, lack of action may prevent the attainment of a desired accuracy for a project, even if all major requirements have been met. The following procedures are precautionary suggestions rather than part of the forma1 specifications.

Collimation and Eccentricity

These are related problems that affect the accuracy of both angle and distance measurements. They may generally be categorized as the centering of instruments and targets. The effect of erroneous centering on distance measurements is direct and rather obvious. For angulation, however, the importance (especially on short lines) cannot be overlooked. With reasonable care, angles can be measured to a precision less than one second over lines of any length and an accuracy (3 sigma) approaching this value for lines in excess of 400 m. With special care, and in controlled situations, angles accurate (3 sigma) to one-half second and less are possible. This is meaningless if the instrument or targets are not centered over the marks and sufficient measurements are not obtained to reduce the observations to the true points.

As a general rule, instruments and targets can be positioned within about 1 mm of the true center. An instrument or target may be deliberately or accidentally operated or observed in an eccentric position, which is usually a short distance from the true station. Accurate measurements of the angle and distance involving the eccentric and true points must be obtained in order to reduce the observations to the true point or to maintain the relationship of the points. The following examples give some indication of the magnitude of angular errors caused by small miscenterings. A 3-mm error in centering a target causes a 0.4-second error at 150 m and 4 seconds at 1500 m. For traverse points spaced at 150 m, if both the target and instrument are miscentered by 3 mm, the angle error could exceed 16 seconds.

Errors caused by a lack of collimation adjustment in theodolites, astronomic transits, and even precise levels are usually eliminated by symmetrical observations in which the collimation error is introduced equally with opposite signs. This is achieved in theodolites and astronomic transits by observing with the telescope in both the direct and reverse positions, and in precise leveling by balancing the sight distances.

Stability and Rigidity of Supports

In precise surveys the towers, stands, and tripods must be stable. The use of driven stakes or some type of quick setting cement for tripod leg supports may be required. Catwalks, supported away from the tripod legs, might also be necessary under some soil conditions to ensure satisfactory results. Regular instrument tripods are rarely used in first- or second-order, class I surveys.

Phase Error

Phase error that occurs in horizontal angle measurement is attributable to an apparent displacement of a target. Such displacements can be caused by unequal daylight illumination of a target or by lighted targets that are not pointed directly toward the observer. For precise surveys, the use of carefully pointed illuminated targets is recommended for both day and night operations. The effect of phase error is the same as the effect described for centering errors, except it is not often readily apparent. Even when detected, phase error is not easily corrected mathematically.

Refraction

A ray of light bends as it passes obliquely through air strata of different densities. Most refraction of this type is in the vertical plane, but frequently there is a measurable component in the horizontal plane. Horizontal refraction is one of the most uncertain factors encountered in the measurement of horizontal angles. Night observations have occasionally been in error by ten or more millimeters and daylight observations by 20 to 30 millimeters. Corrections for refraction cannot be applied to observations, but a careful reconnaissance, avoiding suspicious topographic features and selecting the best available observing conditions, will usually eliminate or substantially reduce this effect. For example, refraction errors that appear in the form of grazing lines often occur when the instrument's line of sight is parallel to a sloping hillside and the wind is simultaneously blowing away from the slope. Another condition to avoid is the presence of obstructions near the ends of the lines, as they might "bend" the line of sight.

Shelter

Protect instruments from uneven thermal expansion and from wind vibration for precise surveys. Minimum protection consists of a large umbrella to block the sun's rays plus a wind screen if needed. A full tent is recommended for lengthy observation schedules under adverse weather conditions.

Adjustments

Least Squares Adjustments

A least squares adjustment is a rigorous mathematical method for adjusting survey data. It has actually been used by surveyors for a number of years, but was generally implemented only on mainframe computers and was somewhat difficult to handle for the uninitiated user. With the advent of new high-speed, inexpensive personal computers and especially modern software techniques, least squares is now readily available to every surveyor. In addition to producing the best adjustment of field data, least squares provides other benefits not even possible with other adjustment methods. It helps in locating errors in survey data, provides an easy way to plan surveys, and provides a statement on the amount of uncertainty for every point in the network.

Least squares is frequently thought of as being difficult to learn, or not being applicable to "the type of surveys that I do." The fact is that least squares is not difficult to understand once a few basic principles are explained; more importantly, it is applicable to nearly all types of survey work, including the small "regular" job. It does not require you to make major changes in your daily practice, although certain field procedures enhance its power.

As surveyors, we have long recognized that adding extra angle and distance observations adds strength to our surveys and allows for error checking. But we also realize that these extra measurements make the resulting survey computations more complex.

As a surveyor, you know that all measurements contain errors. In fact, a measurement is only an estimate of the true value, which is never really known. Figure 4-4 below shows three types of errors commonly present in surveying data (although strictly speaking, blunders are not error), and three methods for handling them.

Error Type	**Method for Handling**
Blunders (mistakes, recording errors, etc.)	Eliminate
Systematic Errors (EDM calibration, etc.)	Compensate
Random Errors (normal, unavoidable)	Adjust with Least Squares

Figure 4-4

Random errors are small, unavoidable, and an integral part of the measuring process. They are the few seconds difference in angles readings, and the few millimeters difference in distances that are present in every traverse. They are no cause for alarm, except that they must be adjusted correctly, and that is the job least squares does properly.

Least squares simultaneously adjust all field data, even in multi-loop traverses. In a least squares adjustment, the "best" solution is defined as the solution producing the smallest changes to the input field measurements. These changes between the best-fit measurements and the original field data are called *residuals*. Technically speaking, the least squares adjustment method minimizes the sum of the squares of the weighted residuals — hence its name.

The term *weight* tells the adjustment how much influence a measurement should have. In least squares, each observation (distance, angle, etc.) can be given an individual weight.

The weight placed on the measurements might be based on the instrument type, the method of observation (chained or EDM distance), and the skill of the field crew. Low weights can be given to less accurately known field data and greater weights to observations that are more accurately known.

During the adjustment, larger changes will be given to the less accurate data, minimizing the changes to the more accurate data. For example, an angle with short sights can be given a low weight so that it does not influence stronger angles with longer sights. Figure 4-5 summarizes the relationships between weights, precision, and influence on the adjustment.

	Strong Measurement	**Weak Measurement**
Weight	High	Low
Precision	High	Low
Influence	High	Low
Standard Error	Low	High

Figure 4-5

Least squares also provides a complete analysis of the survey, including a list of residuals for all measurements, and a statement on the positional accuracy of each computed point. This analysis can assist in the detection of survey blunders and in the preplanning of surveys to meet specified accuracy requirements.

Least squares provides a number of advantages over other adjustment methods.

- It is mathematically correct for all types of surveys, including traverses, triangulation, trilateration, level circuits, resection, and intersection in any combination.
- It computes a single solution, no matter how complex the survey.
- It does not distort field data, as do some approximate methods.
- It allows for independent weighting of all field observations.
- It allows flexibility during data collection — field data can be collected in any order and configuration.
- It gives a statement of the accuracy of each computed point.
- It helps detect blunders in field data.
- It helps with survey planning.
- It tells a lot about the survey.

Traditionally, cross-ties and extra shots were used mainly to “check in.” In least squares, these redundant shots actually become part of the adjustment, adding strength to the survey. Rather than making the survey solution more difficult, redundancies strengthen the survey, make blunder detection easier, and add more confidence that the adjustment is the “best” solution. Also, additional field data can be added to an existing survey at any time, and the adjustment can be rerun.

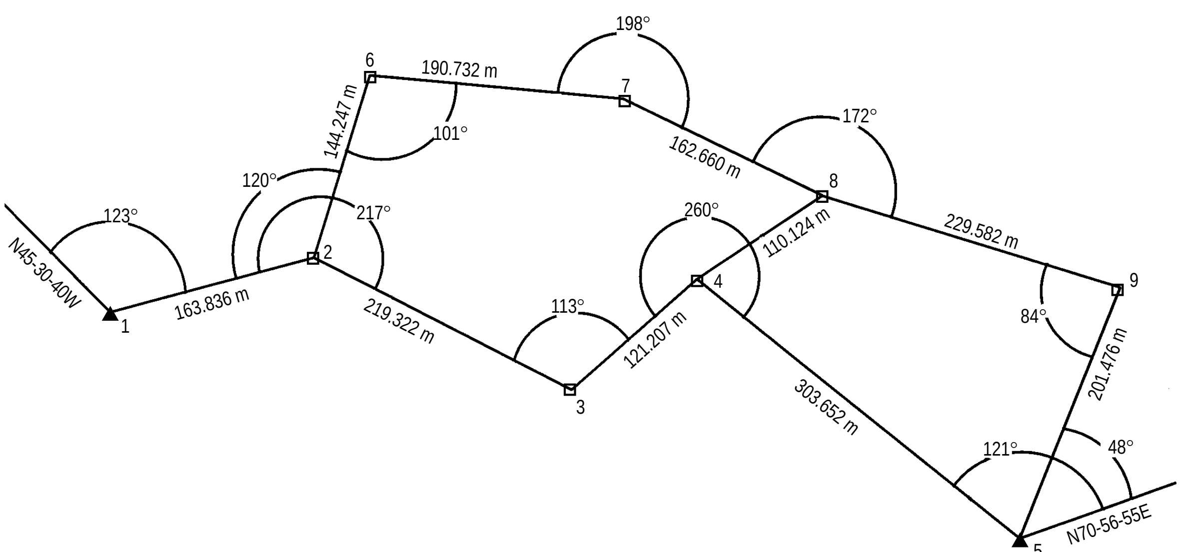
Figure 4-6 illustrates a small survey with two traverse loops and a distance tie between the loops. The two known points have coordinates supplied, and the rest of the field measurements are supplied as angle and distance traverse legs. The sample data field uses a simple code to indicate coordinates (C), traverse lines (TB, TE, and T) and distances (D).

Once the field data has been prepared, a decision must be made as to how observations will be weighted. This is done by establishing a *standard error* for each observation. Standard error is a way of expressing confidence in the field data.

For example, a decision could be made to give the distances a standard error of 6 mm, ±3 ppm, and the angles five seconds. These values are normally determined from instrument specifications and observation procedures. In addition, a centering error of 2 mm may be chosen to account for imprecise instrument centering. This centering error will increase the standard error value for angles with short sights so that they have less influence in the adjustment than those with long sights.

Error ellipses are used to indicate the amount of uncertainty in a computed point’s position, sometimes called the point’s positional tolerance. Least squares, as a part of the solution process, computes how much uncertainty in the coordinates results from the random errors in the field

C	1	304.801	304.801	
C	5	130.186	989.246	
#				
TB	N45-30-40W			# Backsight to fixed bearing
T	1	123-40-28	163.836 m	
T	2	217-11-37	219.322 m	
T	3	113-53-15	121.207 m	
T	4	260-19-24	303.652 m	
TE	5	121-22-46	N70-56-55E	# End by turn to fixed bearing
#				
TB	1			#Backsight to fixed bearing
T	2	120-11-12	144.247 m	
T	6	101-32-30	190.732 m	
T	7	198-13-09	162.660 m	
T	8	172-07-27	229.582 m	
T	9	-84-32-20	201.476 m	
TE	5	48-20-00	N70-56-55E	# Close to fixed bearing
#				
D	4-8	361.30		# Extra distance tie



Small Survey
Figure 4-6

measurements. These positional uncertainties, as represented by the error ellipses, are also affected by the geometry of the survey.

Two simple cases of error ellipses are illustrated in Figure 4-7. The ellipse dimensions indicate the size of the error region, and the orientation indicates the weaker and stronger directions.

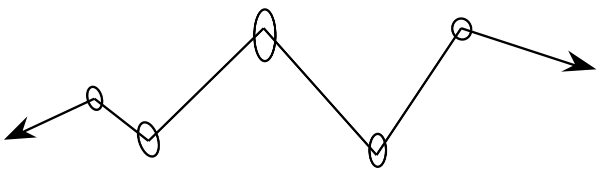


Figure 4-7

In a simple traverse between two fixed points, the error ellipses tend to increase in size according to the point's distance from a fixed station, as shown in Figure 4-8.

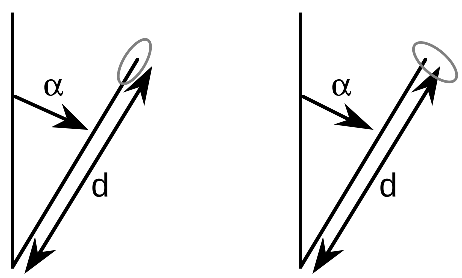


Figure 4-8

Angle measured with high precision. Distance measured with low precision.

Angle measured with low precision. Distance measured with high precision.

Figure 4-9 shows an actual ellipse that resulted from the adjustment of a multi-loop traverse survey. Also shown are the ground dimensions of the error ellipse around the point. Even survey loops that close with very high precision may have large ellipses around the points, depending on the geometry of the survey.

Take the example of the surveyor who traversed through several miles of forest to discover that his newly located section corner was 150 mm away from a monument he found. When he traversed back, he closed to 1:55 000 — so, should the corner be reset? A least squares adjustment of the survey shows that the error ellipse for the new corner was over 450 mm long. This ellipse obviously raises some doubt about whether the new point is really any better than the existing monument (see WAC 332-130-080 Relative accuracy - Principles.).

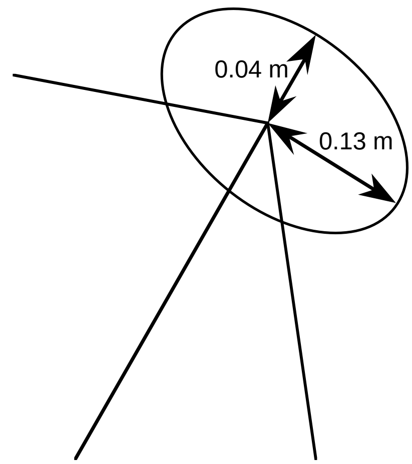


Figure 4-9

WAC 332-130-090 Field traverse standards for land boundary surveys. (effective 4/1/90)

The following standards shall apply to field traverses used in land boundary surveys. Such standards should be considered minimum standards only. Higher levels of precision are expected to be utilized in areas with higher property values or in other situations necessitating higher accuracy.

(1) Linear closures after azimuth adjustment.

(a) City — central and local business and industrial areas 1:10,000

(b) City — residential and subdivision lots 1:5,000

(c) Section subdivision, new subdivision boundaries for residential lots and interior monument control 1:5,000

(d) Suburban — residential and subdivision lots 1:5,000

(e) Rural — forest land and cultivated areas 1:5,000

(f) Lambert grid traverses 1:10,000

(2) Angular closure.

(a) Where 1:10,000 minimum linear closure is required, the maximum angular error in seconds shall be determined by the formula of $10\sqrt{n}$, where n equals the number of angles in the closed traverse.

(b) Where 1:5,000 minimum linear closure is required, the maximum angular error in seconds shall be determined by the formula of $30\sqrt{n}$, where n equals the number of angles in the closed traverse.

Transit and Compass Rule Adjustments

Conventional traverses have traditionally been adjusted by two approximate methods. They are the compass rule and the transit rule. The transit rule is based on the assumption that the angles in a traverse are measured with a higher degree of precision than are the lengths of the sides. It was adopted at a time in history when transits replaced the compass and distances were still being chained. According to the transit rule, the correction to the latitude of a line is to the latitude of that line as the closure in latitude is to the sum of all the latitudes, regardless of sign; and the correction to the departure is to the sum of all the departures, regardless of sign. Although the transit rule is valid for selected lines in a given traverse, it is usually contrary to the assumptions made. Since the compass rule is easier to apply and is valid in most traverse adjustment, there is little justification for use of the transit rule.

The compass rule was adopted at a time when instrumentation had greatly improved. Transits eventually were replaced by theodolites and electronic measuring devices became common. Assumptions were made that with the advent of precision length measurement that distances held an edge over the angles. According to the compass rule, the correction to the latitude of a side is to the length of that side as the closure in latitude of the traverse is to the total length of the traverse; and the correction to the departure of a side is to the length of that side as the closure in departure of the traverse is to the total length of the traverse. This method has served well until the introduction of faster methods of computation.

GPS

Introduction

The development of the Global Positioning System was a significant technical advancement in the Surveying and Mapping industries during the 1980s. GPS proved that satellite-based measurement techniques could compete directly with conventional surveys for many tasks. In many cases GPS Surveys proved significantly faster, cheaper, and more accurate than conventional surveys.

The major limitations to GPS in the 1980s were the lack of satellite coverage, the size, cost, and power consumption of receivers, and the lack of knowledge of data collection and processing techniques.

The size, cost, and power consumption of GPS receivers has also been significantly reduced in the recent years. Receivers have been designed to be easier to use, to provide more information on data quality, and to be more portable. They now compete in size with total stations.

Another significant advancement has been in satellite data collection and post processing techniques. New processing algorithms have been developed to greatly increase the production and efficiency of the system.

Measurement of Carrier Waves

GPS satellites continuously broadcast complex signals that are comprised of carrier waves, codes, and a broadcast message. The basis of the signal structure are the L1 and L2 carrier waves that have a 19 cm and 24 cm wave length respectively.

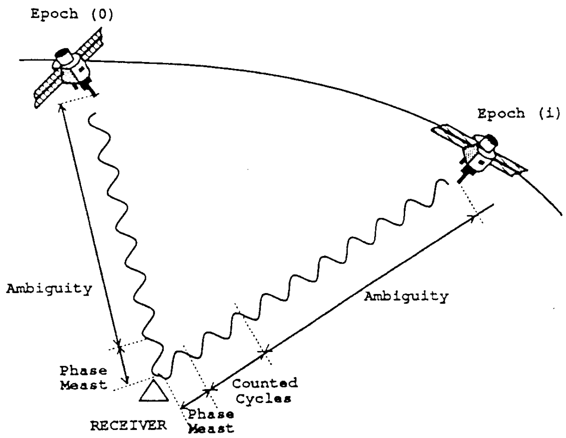
The C/A (Course Acquisition) and P (Precise) codes are modulated onto these carrier waves and have wave lengths of 300 m and 30 m respectively. The codes provide information needed for a receiver to calculate point positions to approximately 50 m accuracy in real time. The broadcast message contains information about the satellite ephemeris, the behavior of the satellite clock and ionospheric models. Engineering, geodetic, and surveying applications call for accuracies in the order of a centimeter or even better. Such precision can only be obtained by measuring the carrier waves from the satellites. In addition, differential observation techniques must be employed.

The carrier wave signal that is broadcast from a satellite and received on earth can be divided into two components — the phase measurement of the carrier wave and the ambiguity.

The phase measurement is the phase of the last incoming wave from the satellite, and is measured directly by the receiver.

The ambiguity is the unknown integer number of carrier-wave cycles between the satellite and the receiver at the starting epoch. The ambiguity value cannot be measured directly and must be calculated.

Figure 4-10 shows the satellite signal at two epochs. When the receiver first locks onto the satellite signal (Epoch O), it measures the phase of the carrier wave, but does not know the ambiguity value. As the receiver continues to track, it keeps count of the integer number of wave lengths that pass by since initial lock-on. At a later time (Epoch i), the receiver measures the phase and counts the cycles since lock-on. The ambiguity value, which is still the same as initial lock-on, is still unknown.

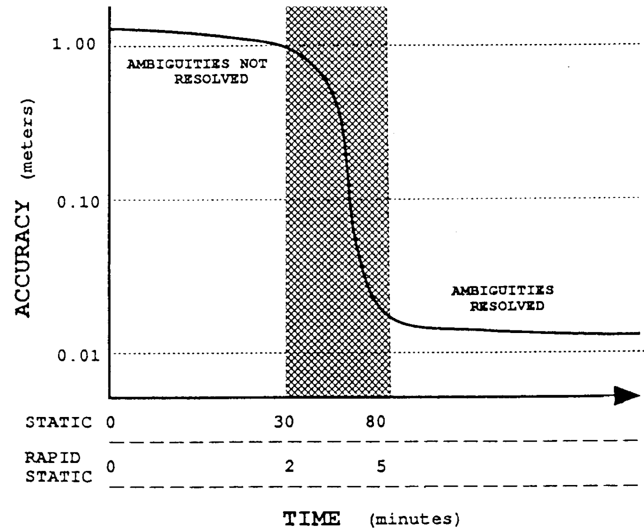


Satellite Signals
Figure 4-10

Resolving the Ambiguities

The correct calculation of the number of wave lengths between the satellite and the receiver is known as "re-solving the ambiguity." The key to high precision results in GPS surveying is to resolve the ambiguity value to each satellite. Until the ambiguity is resolved, the range to the satellite cannot be measured accurately, as the exact number of carrier wave lengths between the satellite and the receiver is not known. Once the ambiguity is resolved, however, the range to the satellite can be calculated almost to millimeter level, and the relative position between stations to the same level of accuracy.

Figure 4-11 shows the affect that resolving the ambiguity has on the results. Before ambiguities are resolved, the GPS user can expect position accuracies around the meter level. Once the ambiguities are resolved, then millimeter accuracies can be achieved. Once the ambiguities are resolved there is very little gain in measuring for longer periods.



Ambiguity
Figure 4-11

The major limitations to resolving ambiguities are disturbed or changing atmospheric conditions and obstructions to the satellite signals. Both of these factors corrupt data and reduce the accuracy as a result. It is usual to collect extra data over long lines where there is more chance of the atmospheric conditions being different at either end of the line.

Traditionally GPS processing algorithms have required at least one hour of data (depending on atmospheric conditions, number of satellites observed and the length of line) to resolve the ambiguities.

The observation time allows the geometry of the satellite constellation to change sufficiently for the ambiguities to be correctly estimated and provides enough data to overcome any problems that might be caused by cycle slips and atmospheric disturbances.

The need to collect sufficient data to resolve ambiguities has limited the speed of GPS survey. Even with this limitation, GPS has still been found to be a productive and competitive measurement technique for long distances and large areas. Up until now, conventional survey techniques have been more cost effective over short distances.

Other Solutions — Other Limitations

Over the last few years there have been a number of attempts to improve the productivity of GPS surveying. These attempts include the kinematic and reoccupation (or pseudokinematic) techniques.

The kinematic technique requires the operator to resolve the ambiguities at the beginning of the observing session. Initial ambiguities can be resolved using traditional GPS Survey methods (occupy each station for about 60 minutes), by beginning the survey on stations with precisely known coordinates, or by using an antenna swap technique. The antenna swap requires that the antennas of both the remote and master station are placed close together and then are swapped over in position.

After the ambiguities have been resolved the operator is able to commence the survey.

One reference receiver stays fixed while the operator moves the roving receiver between stations. Less than a minute of data is required at each station to determine a highly accurate GPS position. It is important, however, that lock is maintained on the satellites at all times. If lock is lost, the survey must start again by resolving ambiguities. The kinematic technique has proved productive and useful for establishing dense control in small areas where there are no obstructions.

The reoccupation or pseudokinematics technique requires that each station is observed at least twice at different times. Lock does not have to be maintained on the satellites between station occupations. The collected data from each occupation is combined in the postprocessing software, which uses the change in satellite geometry to assist in resolving the ambiguities. This method has the advantage that each station occupation need only be a few minutes, but has the disadvantage that every station has to be observed twice. When access to stations is difficult, the pseudokinematic solution is of little practical benefit.

Rapid or Fast Static

The rapid or fast static technique is designed to accelerate the traditional static surveying process to compete with kinematic surveys. This is achieved by using processing algorithms to resolve satellite ambiguities with only a few minutes of data. This technique enables fast, high accuracy positioning, without the limitation of keeping lock on the satellites while moving between points.

The concept of rapid or fast static positioning has been developed by a number of research organizations. These approaches are based on the use of simultaneous phase and P-Code measurements observed on both the L1 and L2 frequencies. Although these approaches enable ambiguities to be resolved in very short time periods, they require a P-Code signal on both frequencies. At present this requirement can be met; however, it could happen that the P-Code signal will be fully encrypted in the future.

It was imperative that industry find an alternate method of ambiguity resolution and not be P-Code dependent. One approach is to use the carrier phase component of the satellite signal, utilizing the statistical and geometrical information provided by the initial differential position adjustment. This information includes the solution vector, the variance-covariance matrix, and the subsequent standard deviation of the unit weight. The task to be accomplished is now to search for each individual real-valued ambiguity parameter and the corresponding integer value. One has to bear in mind that millions of such combinations might have to be analyzed. Therefore, the primary objective is to reduce the number of ambiguity combinations to be analyzed. A dramatic reduction of the search range can be achieved if simultaneous L1 and L2 phase measurements are available. Due to the special geometrical conditions as well as the cancellations of receiver clocks, combinations of L1 and L2 ambiguities can be estimated on a very high level of precision. The achievable reduction of the search ranges can go as far as 0.25 m using combinations. These significant reductions to the integer ambiguity combinations open up the possibilities to deal with very short observation periods.

Ionospheric Disturbances

Undoubtedly, the most limiting factor for positioning with GPS is in heavy disturbances caused by a rapidly changing ionosphere. Loosely speaking, electromagnetic signals are delayed when propagating through the ionosphere, a layer of the earth's atmosphere from about 90 kilometers up to 1000 kilometers containing free electrons and ions. The magnitude of the delay is determined by the number of free electrons along the signal path.

There are three major possibilities to deal with ionospheric disturbances in analyzing GPS data. The first is to correct the GPS observations using a heuristic or even a parametric model of the ionosphere. The success of such corrections is dependent on how well the mathematical model represents the actual ionosphere and on the quality of the data used to derive the model. The second approach to reduce the effects of ionosphere disturbances is to difference observations between sites. This approach cancels the errors common to both sites. The third one is to form so called ionosphere-free linear combinations between L1 and L2 observations making use of the natural dispersing of the various frequencies in the ionosphere.

Usually the first two methods are employed to correct the effects of ionospheric disturbances for short base lines, whereas, the third method is employed for medium to long-range base lines, mainly because of an increase in noise by forming linear combinations between the basic observables.

P:HSM4

The following instructions are for performing and adjusting a traverse using a total station and a data collector.

TRAVERSE CHECKLIST

OBTAIN PROJECT CONTROL

See Geographic Services
Research county records
Existing control from other WSDOT projects

CONVERT CONTROL TO PROJECT DATUM

Discuss this process with the project designer
Use the hand method or the Washington State plane to Project Datum program
Document all calculations and final Combined Factors

DETERMINE ORDER OF ACCURACY FOR TRAVERSE

Refer to the WSDOT Highway Surveying Manual (M 22-97)

LAYOUT THE TRAVERSE ROUTE

Refer to the WSDOT Highway Surveying Manual (M 22-97)
Adhere to the distance and angle requirements for your order of accuracy
Establish solid points
Set points with safety and ease of occupation in mind

SET UP DATA COLLECTOR AND INSTRUMENT

Refer to the SOKKIA SDR 33 Reference Manual
Set appropriate units of measure in data collector
Set up job in data collector
Define the set collection options in the data collector

RUN TRAVERSE

Keep clear accurate notes either electronically or on paper
Always include a sketch of your traverse
Provide accurate descriptions of traverse point locations with reference ties to other objects

ADJUST TRAVERSE DATA

Use Least Squares or Compass Rule adjustment
Document all adjustment calculations

SET COLLECTION WITH THE SDR 33

SET UP A NEW JOB IN THE SDR 33

CHOOSE METRIC UNITS

CHOOSE THE "SURVEY" MENU

CHOOSE THE "KEYBOARD INPUT" OPTION

Enter the coordinates of the known points, then clear when done

CHOOSE THE "SET COLLECTION" MENU

CHOOSE THE "OPTIONS" SOFTKEY

SELECT THE FOLLOWING OPTIONS:

Method	Direction
Data	HVD
Number of H sets	4
Num dist read	1 *(This option only available for SOKKIA instruments)*
Face order	F1F2/F1F2
Obs order	123..321
Return sight	Never *(Select "Prompted" for this class)*
Pre-enter points	Yes

For a description of each option for set collection, refer to chapter 11 of the SOKKIA SDR 33 Reference Manual. This set of options will work well for most situations, but the set collection options can be changed to fit your specific needs.

PRESS "OK"

CONFIRM ORIENTATION

Enter instrument point
Enter ATM data
Enter instrument height
Enter Backsight point

PRE-ENTER POINTS

The BS point will be displayed at the top of the list
Enter all FS points to be observed from current station
When the list is complete, press "OK"

BEGIN SET COLLECTION

Sight BS with Face 1
Sight FS with Face 1
Invert Telescope and sight FS with Face 2
Sight BS with Face 2
When a set is collected within the specified tolerances, select "Change Station" and move to the next traverse point

TRAVERSE ADJUSTMENT WITH THE SDR 33

SELECT "TRAVERSE ADJUSTMENT" FROM THE SURVEY MENU

Enter the point number of the first occupied station
The SDR 33 will then present the traverse route. It will stop when:
It reaches the end of the route
It reaches a branch in the route
It reaches a fixed station (one with known coordinates)
It reaches a fixed point that closes the traverse
It loops to the initial traverse point
200 traverse stations have been exceeded
Press "OK" if the route is complete
If the route is not complete, enter the next point number
The SDR 33 will add more points to the route
Press "OK" when the route is complete

ACCEPT OR EDIT THE TRAVERSE ORIENTATION AND PRESS "OK"

The SDR 33 calculates the precision of the traverse and displays the results

IF YOU WANT TO VIEW THE TRAVERSE, PRESS "VIEW"

Press "CLEAR" to return to the traverse precision screen

PRESS THE "STORE" SOFTKEY TO STORE THE PRECISION RESULTS

PRESS THE "OPTIONS" SOFTKEY TO SET THE ADJUSTMENT OPTIONS AND CHOOSE THE FOLLOWING:

Method	Compass
Angular	Weighted
Elev	Weighted
Report angle adjust	Yes

Press "OK" to accept the adjustment options

TO ADJUST THE TRAVERSE, PRESS THE "ADJUST" SOFTKEY

The SDR 33 will display the adjusted traverse data after angle adjustment

PRESS THE "STORE" SOFTKEY TO STORE THE RESULTS

PRESS THE "ADJUST" SOFTKEY TO END THE ADJUSTMENT AND RETURN TO THE SURVEY MENU

5

Vertical Control

National Vertical Control Network

The National Vertical Control Network consists of a hierarchy of interrelated nets which span the nation. The specifications starting on page 5.27 summarize the classification of the various components. The most crucial elements of this net are:

- A set of large circuits (loops) at an average grid size of 100 to 300 Km.
- A subdivision of circuits with an average grid size of 50 to 100 Km.
- A secondary densification net with a line spacing of 10 to 50 Km. This net provides additional vertical control points, which are determined using slightly lower accuracy standards.

Each level of the hierarchy is adjusted to conform to net elevations having equal or higher accuracy. Elevations computed from a new survey cannot be given a classification higher than that of the elevations which were fixed to define the datum relationship in the adjustment. These primary and secondary nets provide a common reference system for the user's needs. The total net is shown in Figure 5-4.

A continuing program of mark maintenance is needed to replace destroyed marks, relevel older lines of the primary net, and extend control into areas having inadequate coverage. This activity maintains the national vertical control net at a level of accuracy and availability consistent with users' needs.

Elevations in the past have been referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29). The 1929 datum was established by constraining the combined United States and Canadian first-order leveling nets as they existed in 1929, to conform to mean seal level (MSL) of various tidal epochs. These epochs were determined using data gathered from 26 long-term tide stations along the Atlantic and Pacific coasts and Gulf of Mexico.

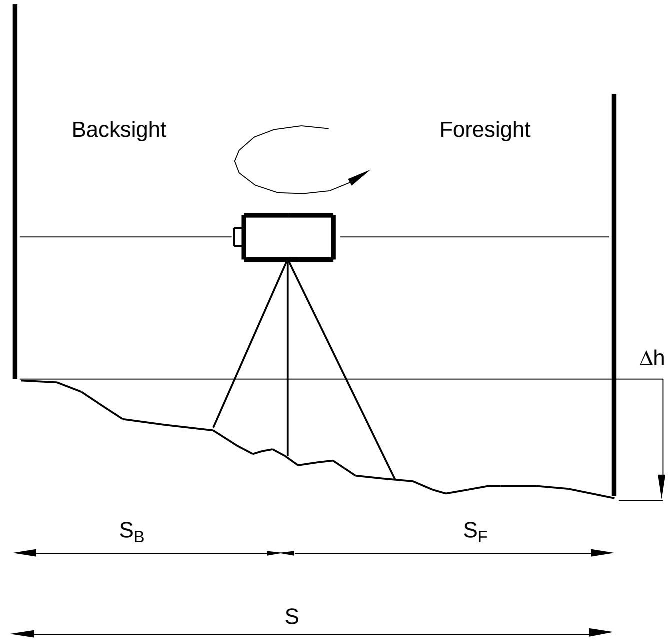
By 1980, a total of 700 000 km of new first and second order leveling had been adjusted into this system.

Today a new datum, North American Vertical Datum of 1988 exists. The NAVD 88 heights are the result of a mathematical least squares general adjustment of the vertical control portion of the National Geodetic Reference System and include 80 000 km of new U.S. leveling observations undertaken specifically for this project. This adjustment was performed with the assumption that the network was on an equipotential surface and it fixed only the vertical control on the Atlantic coast. This had the effect of raising elevations a little over a meter in Washington State as the adjustment was allowed to float. NAVD 88 provides a modern, improved vertical datum for the United States, Canada, and Mexico.

The Federal Geodetic Control Subcommittee (FGCS) has affirmed the North American Vertical Datum of 1988 (NAVD 88) as the official civilian vertical datum for surveying and mapping activities in the United States. WSDOT has adopted the datum.

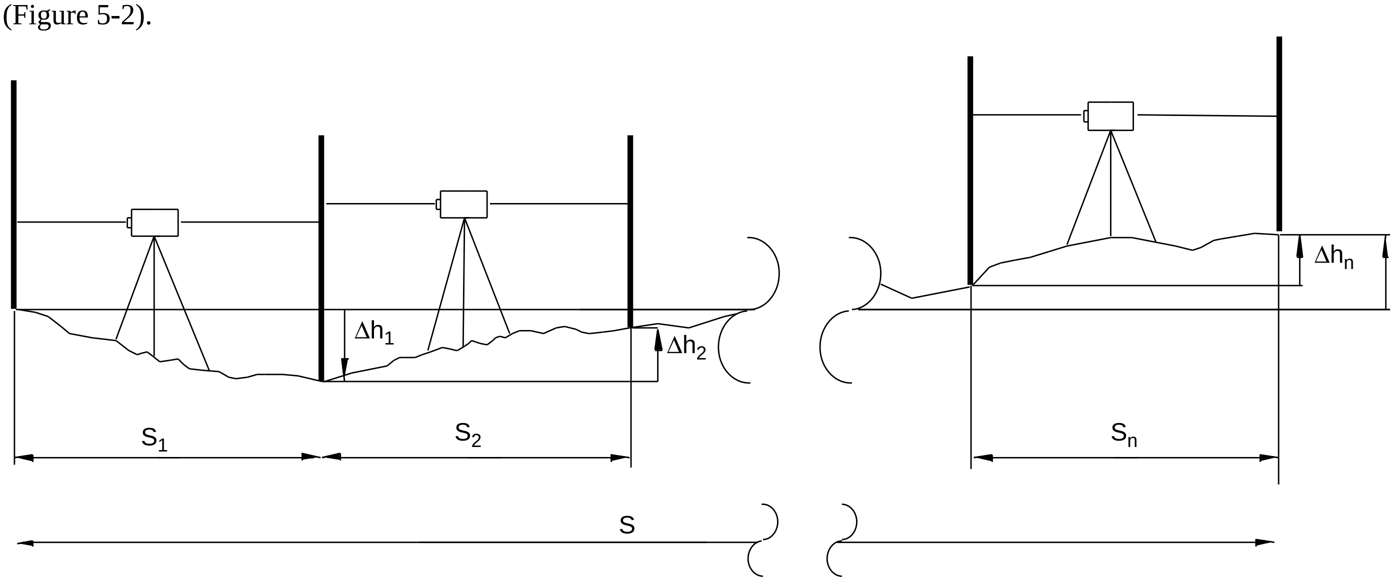
In a widespread network of vertical control, geodetic leveling is the technique that provides the most reliable elevation differences between control points. It is a form of precise leveling where the observing team limits the magnitude of error by using calibrated instruments in combination with a rigorous, symmetrical observing procedure. In the following pages leveling procedures are described, the principal sources of error are discussed, and error tolerances are presented to provide a foundation for the instructions that comprise the rest of this chapter.

Along each line of a vertical control network, leveling is conducted in increments called sections. Each section is an unbroken series of setups, made between two permanent control points. A setup consists of a point supporting a backsight rod, a point supporting a foresight rod, and a leveling instrument positioned between them (Figure 5-1). Two heights are measured by sighting through the instrument toward a scale on each rod and recording the values intercepted by a line on the reticle. The height difference, backsight minus foresight, corresponds to the elevation difference between the two points. The foresight point of one setup becomes the backsight point of the next; thus, the sum of the elevation differences of the series of setups is the elevation difference for the section (Figure 5-2).



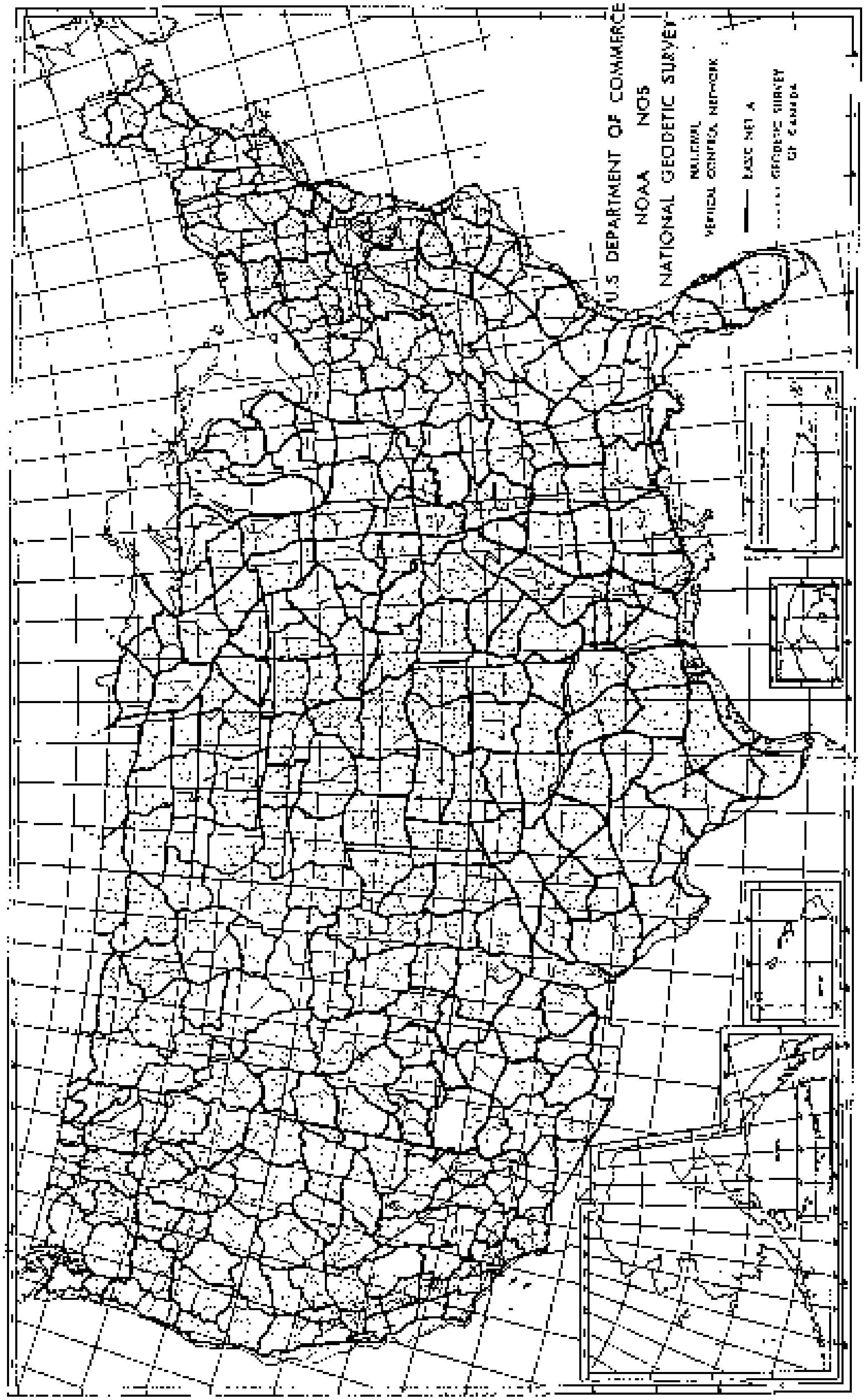
$\Delta h = B - F$ and $s = s_B + s_F$

Setup of Leveling
Figure 5-1

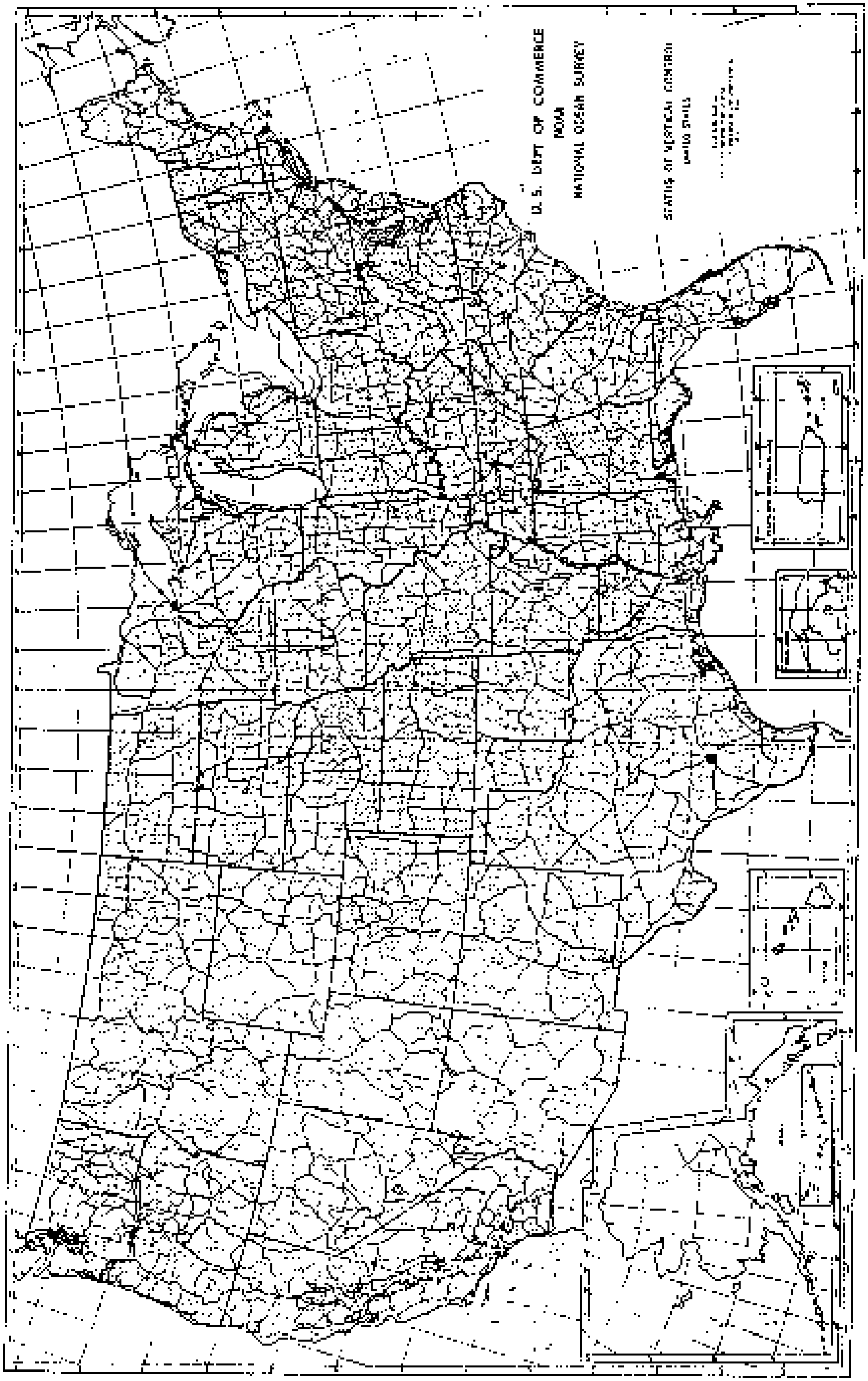


$\Delta H = \Delta h_1 + \Delta h_2 + \ldots + \Delta h_n$ and $s = s_1 + s_2 + \ldots + s_n$

Section of Leveling
Figure 5-2



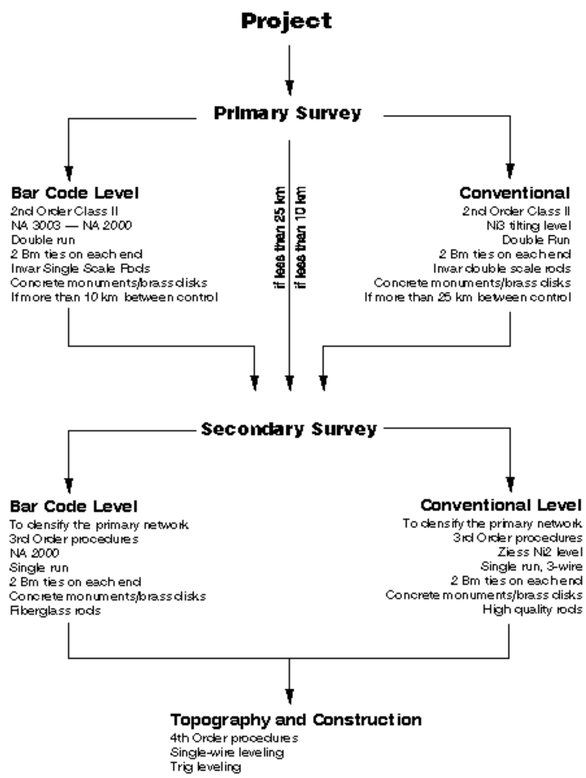
Basic Net A of the National Geodetic Vertical Control Network
Figure 5-3



United States Vertical Control Network
Figure 5-4

Three conditions must be satisfied for this technique to provide reliable elevation difference between points. First, the lines of sight from the instrument to the rods must be level; in other words, the lines of sight must be parallel at all times to the reference surface. Second, the values observed on the scales must accurately indicate heights above the points on which the rods rest, and third, the points in turn must be stable with respect to the topography.

These conditions cannot be perfectly satisfied in the "real world". However, they may be approximated by limiting the known sources of error. Leveling is classified by the degree with which error magnitudes are limited.



Project Flowchart
Figure 5-5

Most WSDOT projects require leveling in the range of third and fourth order of accuracy. However, higher orders are sometimes required for primary control. All classifications are shown in this section to get a "feel" for what is involved. Use Project Flowchart Figure 5-5. A suggestion would be to use a bar-code level for all Primary and Secondary Leveling projects as single wire procedures are acceptable only for fourth order.

First-Order, Class I

First-order, class I leveling is primarily used for basic net A. It is also used for metropolitan area control and regional engineering. Line spacing for basic net A is 100 to 300 km, depending on population density and use. The maximum length of line between net junctions is 300 km. Bench mark spacing averages 1.6 km. Maximum separation does not exceed 3 km. Closer spacing is appropriate for more densely populated areas. Bench mark spacing is reduced to less than 1 km along steep slopes in order to avoid setting temporary bench marks. Sections are observed either in both forward and backward directions, or in one direction when using the double-simultaneous mode. Gravity values are required at bench mark settings if elevation differences and elevations are to be computed in geopotential units (gpu). Gravity values can either be observed or interpolated from nearby observed gravity, but the effect on geopotential heights is not allowed to exceed the limits stated in the Federal Geodetic Control Subcommittee (FGCS) specifications at the end of this chapter.

First-Order, Class II

First-order, class II leveling is most often used in establishing basic net B, and for releveling portions of basic net A. It is also employed for metropolitan area control and regional engineering. Line spacing on basic net B is between 50 to 100 km, with a maximum distance between junctions of 100 km. Mark spacing, instrumentation, section runnings, and gravity requirements are provided in the FGCS specifications at the end of this chapter.

Second-Order, Class I

Vertical surveys of second-order, class I accuracy are used for the secondary net, for metropolitan area control, and for large engineering projects. Line spacing is 20 to 50 km, with a maximum distance of 50 km between junctions. Instrumentation requirements are the same as for first-order leveling, except three-wire instruments and single-scale invar rods can be used. Section runnings and gravity requirements are provided in the FGCS specifications at the end of this chapter.

Second-Order, Class II

Second-order, class II leveling is used for area control, local engineering, and topographic mapping projects. Line spacing for area control is 10 to 25 km. The maximum length of line between junctions is 50 km for double-run and 25 km for single-run leveling. The maximum line length can be increased to 100 km for double-run leveling in areas where higher order control nets have not been established. Geodetic level instruments and invar rods shall be used for double-run leveling. Sections may be single-run (if less than 25 km) or double-run (if greater than 25 km). Gravity requirements are provided in the FGCS specifications at the end of this chapter.

Third-Order

Third-order leveling is used for local control, small engineering projects, topographic mapping projects, and other work requiring this accuracy. Line spacing depends on project requirements. The maximum length of line between two junctions is 25 km for double-run and 10 km for single-run leveling. The maximum length of a double-run line may be increased to 50 km in areas where higher order control nets have not been established.

Fourth-Order

Connections must verify observed elevation differences of established bench marks within the prescribed limits of the lowest order survey of previous or new leveling; that is, a third-order survey must check between two marks of a previous first-order survey to third-order tolerance limits. Check connections shall be single-run unless an error is suspected in the new leveling. In first- and second-order check connections to previous first- and second-order surveys, NGS, or the originating agency for the previous survey, should be consulted if satisfactory checks are not obtained after tying the survey to one mark greater than the number required for the check. (See Figure 5-6.) Failure to check previous leveling can be caused by crustal motion, physical disturbances, or geological phenomena. Checking the elevation difference between two bench marks located on the same structure, or so close together that both may have been affected by the same localized disturbing influence, is not considered a proper check. If the elevation difference between two consecutive bench marks does not check, but the sum of the two "new and old" differences agrees within stated tolerance limits, it can be accepted as a "two bench mark" check ("jump" check). (For example, see Figure 5-7.)

Sources of Error

Error may be characterized as random or systematic. Random errors in leveling results represent the effect of unpredictable variations in the instruments, the environment, and the procedure of leveling. Random errors cannot be completely eliminated, although they can be kept small. They are the limits on the precision with which leveling may measure elevation differences.

Systematic error represents the effect of consistent inaccuracies in the instruments or in the leveling procedure. It also results from consistent, though not always predictable, environmental effects. Although systematic error may be small in a single measurement, it accumulates when several measurements are made under similar conditions. Thus, it can result in a significant discrepancy in the elevation differences measured between two control points by different leveling systems and/or routes. For leveling to provide accurate elevation differences, systematic error must be eliminated, either by procedure or by applying corrections to the data.

The sources of error in leveling can be classified into three groups: those affecting the line of sight, those affecting the heights observed, and blunders. The line of sight cannot be exactly level because of the effects of imperfections in the instrument and variations in the human eye, refraction, curvature, and tidal accelerations. The heights observed are not exact because of imperfec-

Classification	Fourth Order
Maximum length of sight	90 m
Maximum difference in lengths of forward and back sights	
per setup	12 m
per section	9 m
Maximum length of line between connections	4.8 km
Closure, maximum loop or line	0.129 km
Instrumentation	Any good quality tripod-mounted engineer's level; wood or alloy rods
Field procedure	Single-run loops closing on themselves or single-run lines between bench marks established by equal or higher classification survey. Only the middle wire need be read if stadia distances are not used to determine lengths of level lines, lengths of sights, or differences between lengths of forward and backward sights.

Classification for Plane Leveling
Figure 5-6

(1)	Title of Job	(2)	Date
(3)	Inst. Type/Type of Rods	(4)	Inst. Person
(5)	Weather		Rodman
(6)	Temp		Rodman

(7)	Station Designation	(8)	Sta. Elev. & Datum
	B 466 1978		122.200 Feet
	C 466 1978		125.260 Feet

Beginning Station	Backsight	HI	Foresight	Elevation
B 466				122.200
	1.100	123.300		
			1.500	121.800
	0.750	122.550		
			0.795	121.755
	1.350	123.105		
			0.354	122.751
	0.500	123.251		
TBM1			1.850	121.401
	2.250	123.651		
			1.88	121.771
	1.89	123.661		
			0.758	122.903
	2.35	125.253		
			1.1	124.153
	1.89	126.043		
C 466			0.75	125.293

Sum of 12.08 8.987

Field Vert Diff = BS - FS = 12.080 - 8.897 = 3.093 feet
Published Vert Diff = (B466 - C466) 122.200 - 125.260 = 3.060 feet

Misclosure Between Control Marks 0.033 feet
Distance Covered = 0.90 Miles or 1.79 Km

Example of Single Wire Differential Leveling
Figure 5-7

tions in the rod scales and the turning points; furthermore, a perfectly stable relationship cannot be maintained between the equipment and the topography because of environmental effects on the equipment. Blunders may occur while attempting to limit any of these errors.

Differential Leveling Instrument

The instrument used for leveling should consistently provide a horizontal line of sight. The extent to which it achieves this determines its suitability for various orders and classes of leveling.

To be horizontal, the line of sight should be perpendicular to the direction of gravity, at the vertical axis of the instrument. If the line of sight is not horizontal, the angle by which it deviates from horizontal causes an error in every observation (Figure 5-8). This angle is referred to as collimation error.

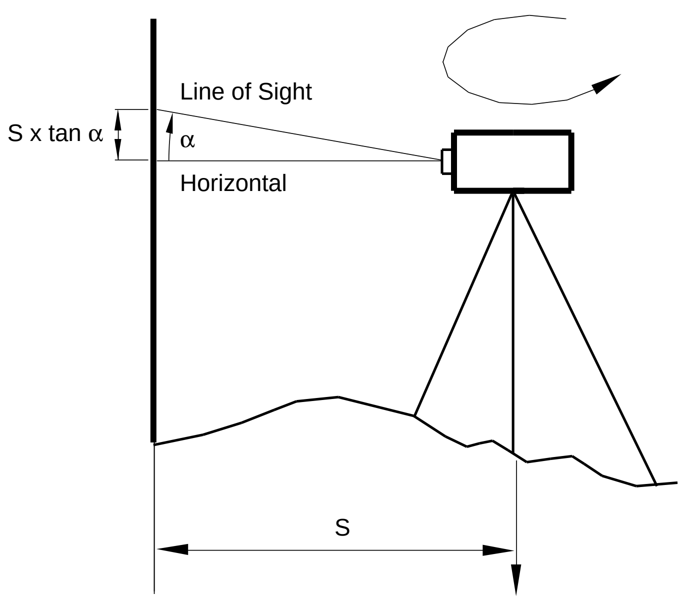
Collimation error can be limited by using a well-designed, and properly maintained instrument. The angle should be measured and adjusted to specifications. The effect of collimation error on each observation can be reduced by limiting the sighting distance. Furthermore, if the sighting distances in each setup are balanced, the errors resulting from collimation error become equal. They cancel when the foresight is subtracted from the backsight to compute the elevation difference (Figure 5-9).

Although it is impractical to balance every setup exactly, the total contribution of collimation error can be limited very effectively while leveling by keeping the imbalance small and random in sign. Any systematic contribution that accumulated with distance may be eliminated by later applying corrections computed from the imbalance and a precisely determined value for the collimation error.

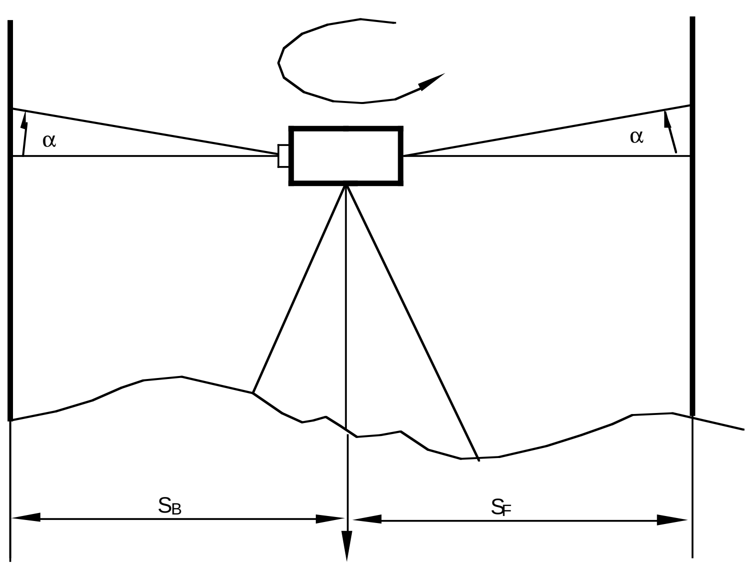
Collimation error should not change as the instrument is refocused or rotated about its vertical axis. A consistent difference between the collimation errors of the backsight and foresight causes a systematic accumulation of error (Figure 5-10). The precautions described here will limit this effect, but, because it is unpredictable, it cannot be eliminated. Additional precautions should be taken to ensure that a consistent difference does not exist.

In a compensator-type leveling instrument such a difference can occur if the compensator is not suspended properly in each direction. To prevent gross error the spherical level on the instrument should be properly adjusted, and compensation checks should be performed routinely. To eliminate smaller systematic effects, the compensator should be repositioned during each setup.

In a spirit-level type of leveling instrument, the error caused by imprecision in centering can become systematical if the bubble is consistently affected by a



Effect of Collimation Error, α
Figure 5-8



Consistent collimation error cancels in a balanced setup since $s_B = s_F$

Collimation Error
Figure 5-9

heat source in one direction. Shading the instrument should reduce this effect.

Pointing

Another effect on the line of sight results from the human inability to repeat a pointing exactly. Both imperfections in the instrument and atmospheric refraction may contribute to this effect, combining with the imperfection of the human eye, to create pointing error.

The magnitude of pointing error is reduced by the use of a precise instrument with a micrometer and wedge reticle that has been adjusted to remove parallax. Limiting the sighting distance and instrument movement can also reduce the magnitude of the error.

Refraction

Variations in atmospheric density cause the line of sight to refract or bend in the direction of increasing air density. These variations seem to be primarily a function of air temperature.

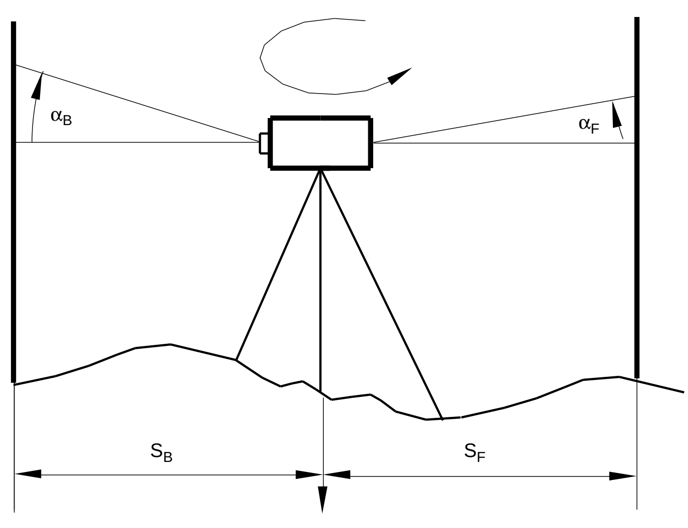
Refraction is most noticeable when the line of sight passes through air of fluctuating density, as when “heat waves” are observed. The graduations on the scales appear to move up and down rapidly. This phenomenon, called shimmer (Figure 5-11), makes it difficult to interpret the scales precisely, thus increasing the magnitude of pointing error. Shimmer can be reduced by shortening the sighting distances or, in some cases, raising the height of the line of sight.

Whether or not shimmer is observed, the line of sight may be refracted. Since the error caused by refraction increases proportionately with the square of the distance, refraction error may be reduced by limiting the sighting distance. As long as atmospheric conditions are similar along both the foresight and backsight, the error may be nearly eliminated by balancing setups.

However, conditions are often not the same along both lines of sight. Air close to the ground varies in density more rapidly than air situated 1 m or more above ground. This can be visualized by imagining air layers conforming to the topography.

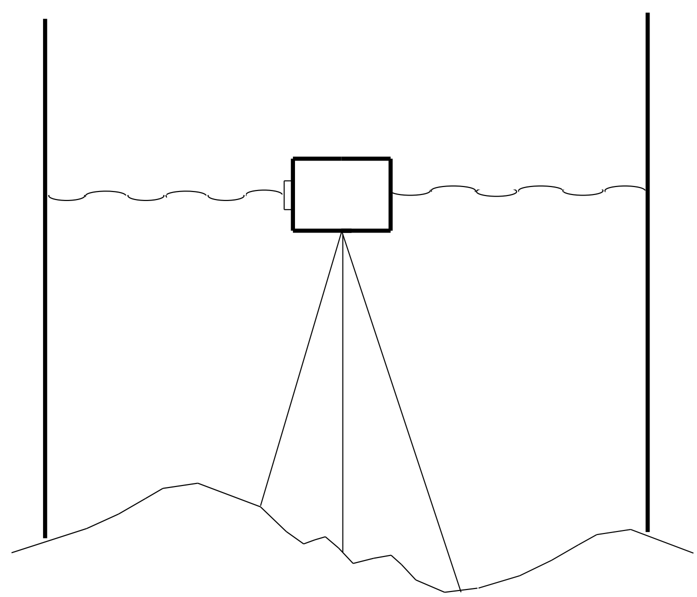
On a slope, even if setups are exactly balanced, the conditions along the foresight differ from those along the backsight (Figure 5-12). Because the sight uphill passes through a greater range of air density, it is refracted more. Refraction error, then, accumulates with change in elevation.

Leveling results may be corrected, at least partially, for refraction if atmospheric conditions are determined and recorded with the observations. Of the many mathematical



Inconsistent collimation error does not cancel in a balanced setup since $\alpha_B \neq \alpha_F$ even if $s_B = s_F$

Collimation Error
Figure 5-10



Shimmer
Figure 5-11

models that attempt to predict refraction error, the most successful require that air-temperature differences be precisely measured during every setup.

Two types of refraction error cannot be corrected. Therefore, the situations which cause them should be avoided when leveling. If the line of sight passes very close to the ground or to an intervening object, changes in air density cause the line of sight to be refracted in an unpredictable way. Similarly, when the air near the ground is cooler than the air above it, the relatively stable air layers may move slowly across the line of sight, causing long-period shimmer. The graduations on the scales appear to move up and down, but so slowly that an entire setup may be observed and checked before the movement is noticed. The observations may be significantly and unpredictably affected by the unnoticed shimmer. This effect usually occurs at night when the air is calm.

Curvature

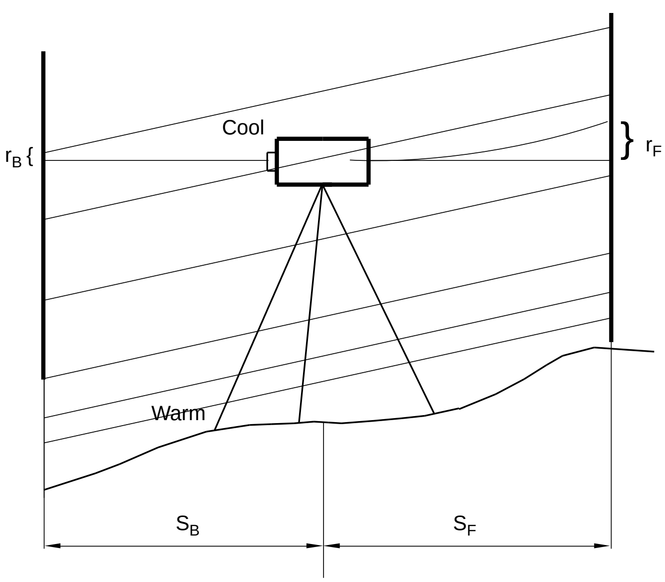
Both the leveling instrument and the rods are oriented to the direction of gravity, to measure elevation differences with respect to the same reference surface. When the instrument is level and is rotated so that the line of sight intercepts each scale, the line of sight should sweep out a horizontal plane. It would be parallel to the equipotential surface if the gravity field, at each setup, also defined a plane. This is not the case, however, since the gravity field defines a curved surface. Thus, a small amount of curvature error is introduced into each observation (Figure 5-13).

Curvature error is proportional to the square of the sighting distance. Assuming the equipotential surface is evenly curved, curvature error can be reduced by making the backsight and foresight distances in each setup nearly equal. If setups are thus balanced, within a tolerance, correction for curvature need not normally be made while leveling.

Because the surfaces defined by the gravity field are not evenly curved, minute differences in curvature error in every setup accumulate systematically in leveling conducted over large changes of elevation, particularly in a northerly or southerly direction. Even so, the magnitude of this error is so small that it may be neglected if the backsight and foresight distances are nearly equal.

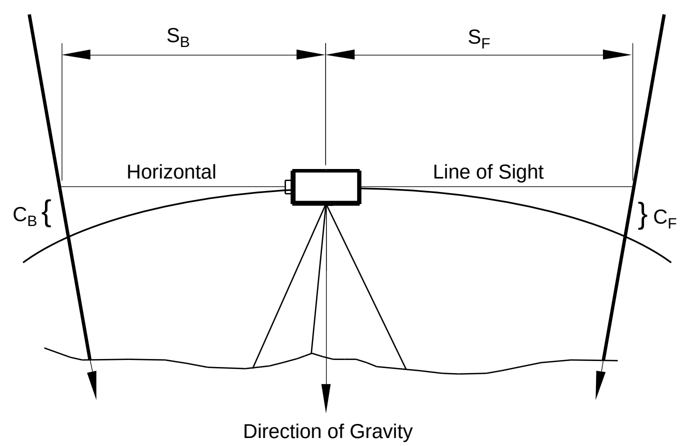
Tidal Accelerations

Because leveling instruments and rods are oriented to the direction of gravity, after curvature has been taken into account the elevation difference of each section is computed along a route that approximately parallels an equipotential surface. However, the sun and moon create



Refraction error, r, does not cancel on sloping terrain

Refraction Error
Figure 5-12



Curvature error, c, where the line of sight is not parallel to an equipotential surface, cancels if $s_B = s_F$

Curvature Error
Figure 5-13

tidal accelerations that periodically distort this surface, generally more toward the equator than the poles (Figure 5-14).

The distortion is termed a deflection and is described by two component vectors. The vertical component affects only the magnitude of gravity along the route, resulting in a negligible effect on the elevation difference. The horizontal component, however, acts at 90° to the equipotential surface, resulting in a small error, especially if the section is oriented in a line with the sun, moon, and the north or south pole. The error is accumulated significantly in leveling lines oriented north-south, particularly in the middle latitudes. To remove it, a correction must be applied.

Leveling rod

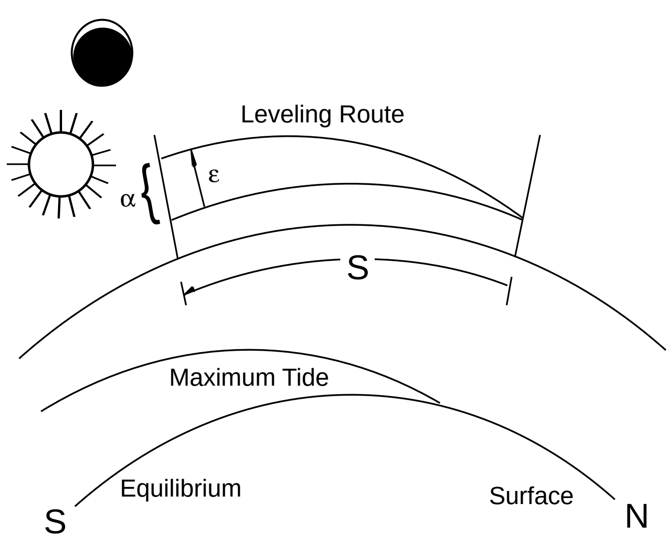
Leveling rods must be carefully designed, precisely manufactured, regularly calibrated, and properly used if they are to provide accurate heights above turning points and control points. In the following pages leveling rods are described, and the requirements for their use, calibration, and maintenance are presented.

Leveling rods have often been made of seasoned hardwood, with a scale printed or stamped directly on the wood. Typically, such a rod is constructed of two or more telescoping sections. The overall length of the scale on a wooden rod changes significantly with normal variations in atmospheric humidity. Furthermore, when assembling two or more telescoping sections, mismatches at the junctions introduce further errors in the scale. Therefore, for geodetic leveling, a much more precise rod is required.

To observe accurate heights above the points on which the leveling rods rest, precise relationships must be maintained between the rods and the equipotential surface, and between the rods and the scales mounted on or within them.

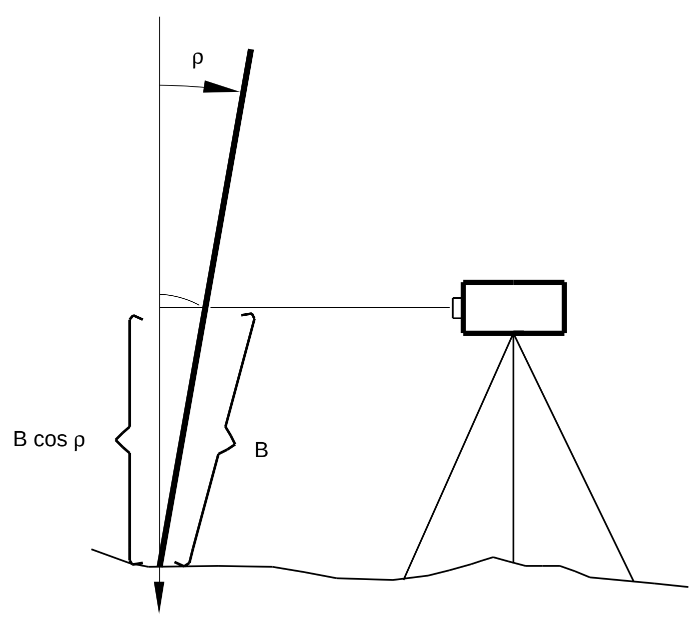
The first relationship is ensured by plumbing, or aligning the rods with the direction of gravity, a task which is analogous to leveling the instrument. If they are not so aligned, an error is introduced into each observation. Although the error may be small, it accumulates systematically with change of elevation, especially on steep slopes where observations are made alternately low and high on the scale. (See Figures 5-15, 5-16, and 5-17.) It can be limited only by plumbing the scales properly.

The second relationship depends upon the accuracy with which the scales are manufactured and mounted in or on the rods. If the graduations are not accurately marked above the scale zero or if the scale changes in length



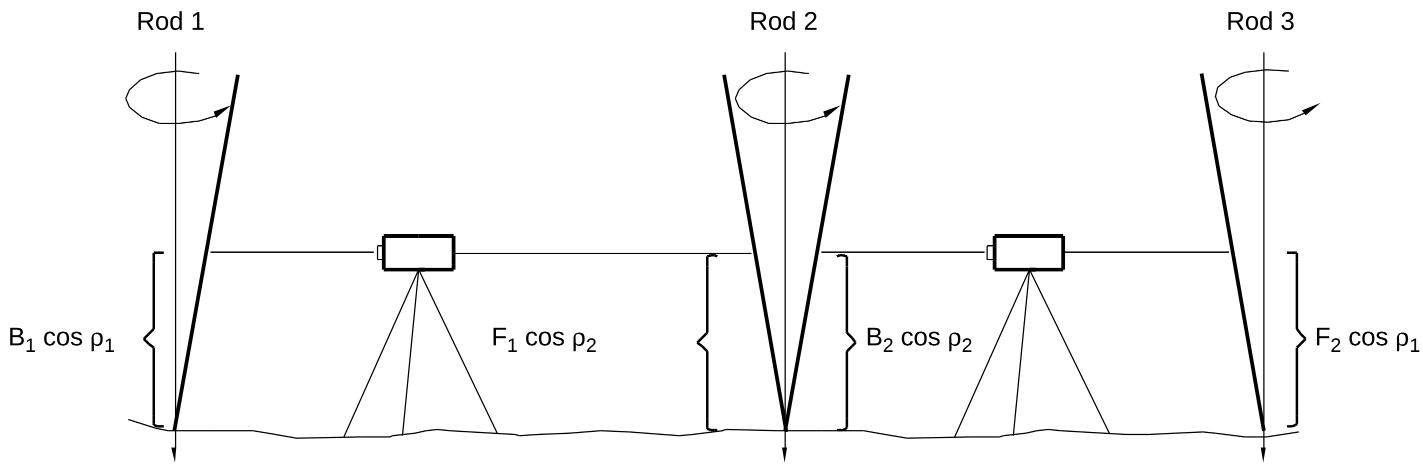
Effect, α, of tidal deflection, ε, on a section of length and direction

Tidal Deflection Error
Figure 5-14



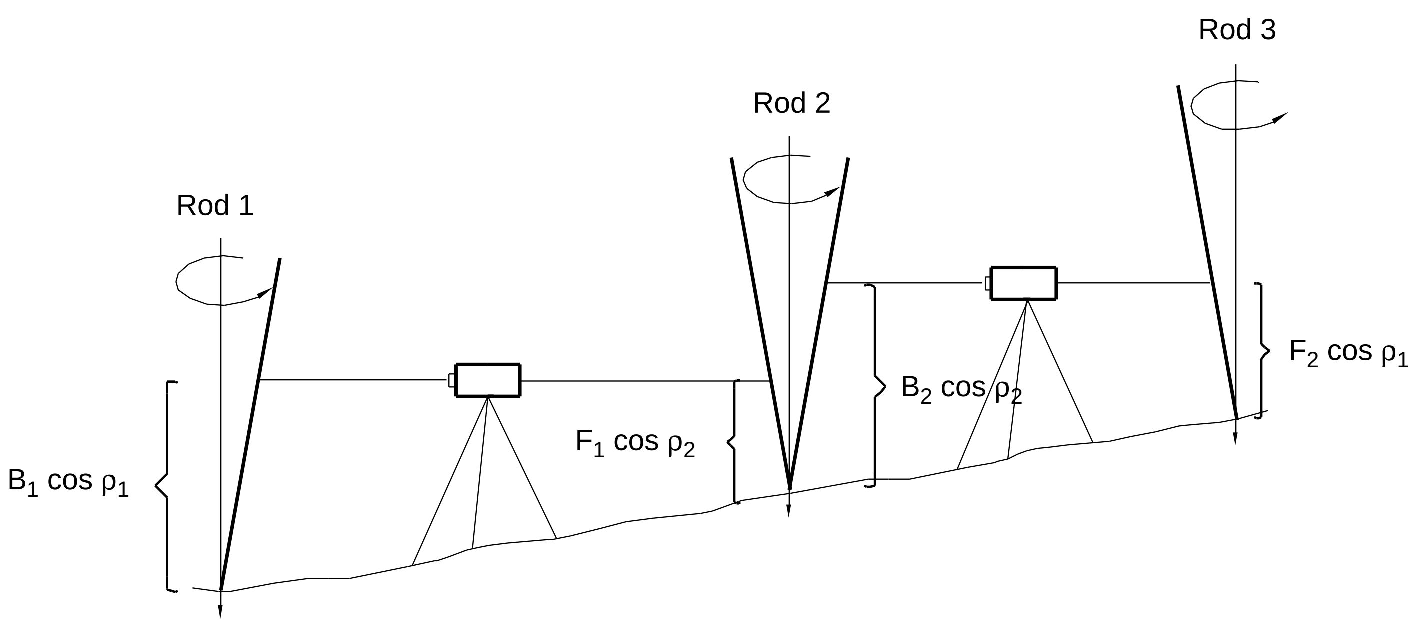
Effect of rod plumbing error, ρ, on a height observation, B

Rod Plumbing Error
Figure 5-15



Systematic effect of plumbing error (and scale errors) is small on flat terrain, since $B_1 \cong F_2$ and $F_1 \cong B_2$

Rod Plumbing Error
Figure 5-16



Systematic effect of plumbing error (and scale errors) accumulates on sloping terrain, since $B_1 \neq F_2$ and $F_1 \neq B_2$

Rod Plumbing Error
Figure 5-17

during leveling, error may accumulate in the observations. To limit the error, leveling rods should be well-designed, routinely calibrated, and properly handled and maintained.

The index error, which is the difference in height from the scale zero to the base plate of the rod, represents a constant portion of the error in the scale values. Index error can be eliminated by making an even number of setups for every section, thus using the same rod on every bench mark during the leveling. It can also be eliminated if only one rod is used if accurate calibration corrections are applied.

In much the same way as an error in plumbing, error in the scale values causes systematic error to accumulate with change of elevation. For the most accurate results, observed scale values should be corrected to standardize them to the National Standard of Length. To do this the rods must be accurately calibrated. Since the scales are subject to thermal expansion, the coefficient of thermal expansion must also be measured, and the scale temperatures should be recorded during the leveling.

In the other procedure, three-wire leveling, the first reading of the setup is made on the backsight of odd-numbered setups, and on the foresight of even-numbered setups. Consistent settlement of the leveling instrument will cause the results of odd setups to be too large and the results of even setups to be too small by a similar amount. Thus, over a section with an even number of setups, the errors nearly cancel. Rod movement is minimized by placing the turning points so they provide sufficient stability. Systematic error caused by consistent movement during each setup can be limited somewhat by the procedures described previously. If the rods are allowed to rest on the points for 20 seconds before making an observation, any remaining movement should be negligible.

When proceeding from one setup to the next, the forward turning point must not move. This can only be assured by the rodman's diligence and by comparing repeated levelings of the section. Averaging the results of the two runnings, made in opposite directions, nearly eliminates any systematic accumulation of error resulting from consistent movement.

Tolerances for Geodetic Leveling

To produce reliable elevations, the results of geodetic leveling must satisfy the appropriate standard of accuracy for the order and class of a survey. This standard is attained in three ways while leveling: first, by operating a well-organized and well-trained leveling crew; second, by selecting sufficiently precise equipment and calibrating it properly; and third, by adopting observing and recording routines that limit accumulation of error. Tolerances, suitable for attaining each standard of accuracy, are presented in the FGCS specifications at the end of this chapter.

Rod Usage

Setting up the rod

Since the base plate corresponds to the zero point of the scale, it must be kept perfectly clean and free from corrosion if precise heights are to be observed. Before setting the rod on a point, carefully wipe away any dust or dirt on the base plate and the point. Do not allow the base plate to rest directly on the ground since scratches may result. A rod with a defective or damaged base plate should be repaired and calibrated.

Set the rod gently onto the point; never drop it. Dropping a rod on a turning point or control point is comparable to hitting the point with a 7 kg sledgehammer. Even the best quality bench mark may suffer from such treatment.

Place the rod so that the exact center of the base plate rests on the highest point of the turning point or control marker.

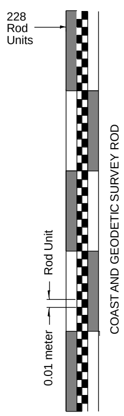
Checking rod plumb

During a setup each leveling rod must be plumb (vertically aligned with the direction of gravity). Achieve this by centering the bubble in the vial of the circular level attached to the rod housing.

Each day, while leveling, the observer should check the plumbing of each rod by comparing the alignment of each rod to the vertical line in the reticle of the leveling instrument. Check the rod while it faces the instrument; then, after the rodman turns the rod 90° (at a right angle to the instrument), check it again. This procedure quickly reveals gross errors in the plumbing of the rod.

For three-wire leveling, rods must have single scales and block graduations. In addition, each rod should have a check scale in different units on the back. Block graduations are the borders between rectangular spaces, which usually alternate black and white in color. Each block is of a height equal to one rod unit. This type of graduation permits the intercept of the line of sight to be estimated to one-tenth of a rod unit. Black blocks must not be thought of as black graduations on a white background.

Figure 5-18 shows the block graduation devised and used by the U.S. Coast and Geodetic Survey from 1916 to 1962. The unit is 1 cm, but the rod appears to have two adjacent, parallel scales. In each centimeter, there are a black block and a white block side by side, causing a checkerboard appearance. The purpose of this scale is to make a white block available, in every centimeter, against which the black reticle line shows distinctly, thus facilitating estimation of tenths of units. This scale must not be confused with the offset parallel scales used for the micrometer-leveling procedure.



Leveling Rod with Block Graduations
Figure 5-18

Turning Points

A turning point is the temporary support on which a leveling rod is placed during a setup. The foresight point for one setup becomes the backsight point for the next, “holding” the elevation while the leveling instrument is moved between setups. Therefore, turning points must be stable, well defined, and properly spaced to measure accurately the elevation differences from one setup to the next. The degree of care taken by the pacer and rodmen in setting and using turning points greatly affects the quality of the leveling. This is especially true in single-run leveling, since movement of the turning points between setups is not easily detected.

Selection and Use

A turning point should not settle or rebound. It should have a definite high point on which to hold the rod and should be easy to set and remove. Do not use objects such as stones, fire hydrants, railroad spikes and ties, or marks on the pavement. Instead, use a pair of standardized points, suitable for the terrain likely to be encountered. Any of the three types described in this section is acceptable. The most satisfactory of these for routine use, under a variety of conditions, is the turning pin. On concrete sidewalks and gravel, the turning plate is suitable. If the route crosses sandy or marshy ground (and the line cannot be relocated to avoid this), a wooden stake with a double-headed nail provides the most reliable turning point.

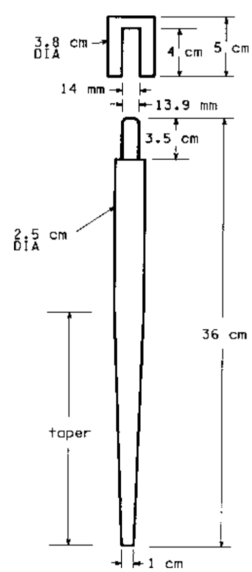
General instructions

When setting a turning point of any type, avoid loose soil, sand, or unpacked gravel. If necessary, dig away the loose layer with a shovel. Avoid frozen ground, but if it cannot be avoided, dig through the shallow frozen layer to unfrozen ground in which a turning point may be placed.

Once set, any turning point can easily settle, rebound, or be displaced. Often these changes are not detectable. Be alert to the possibility of such movement by observing the following precautions: do not drop the leveling rod onto a turning point; do not remove the rod once it has been placed, since removing and replacing the rod can cause a significant change in the elevation of the point; maintain a constant weight on the point by allowing the rod to stand free from downward pressure as much as possible; do not walk around the point unnecessarily.

Turning pin

A turning pin is illustrated in Figure 5-19.



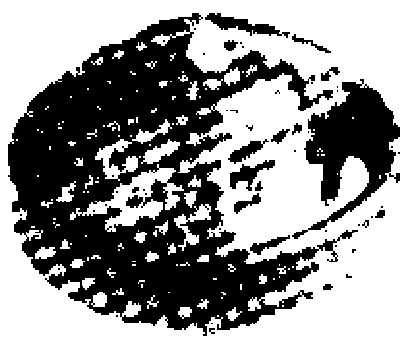
Turning Pin Dimensions
Figure 5-19

To set a turning pin, place the driving cap over the high point and use a 2 kg hammer to drive the pin into the ground. The faces of the hammer should be made of replaceable soft plastic, to prevent damage to the driving cap. Do not hold the hammer with its head crossways when hammering since this may damage the metal part of the head and make replacement of the faces difficult. Drive the pin straight, not at an angle, until the driving shoulder is within 5 cm of the ground. If the pin is set at an angle, or not driven deeply enough, it is more likely to rebound or settle.

After setting the pin, remove the driving cap and carefully place the rod on the high point. After the setup has been checked, gently strike the side of the pin from two directions. This should loosen the pin in the ground so it can be pulled by hand. Avoid pulling the pin by tugging on its cap or chain.

Turning plate

When a turning pin cannot be driven efficiently, such as in the hard-packed gravel of a road shoulder, in very firm clay, or on a concrete sidewalk, use a turning plate (turtle). A turning plate should have (1) a definite high point, (2) three flat feet, and (3) weigh at least 7 kg. Figure 5-20 shows a recommended design.



Turning Plate
Figure 5-20

Exercise caution when setting the turning plate. Simply dropping it onto the ground will not provide enough stability. Tamp the turning plate firmly in place, even on concrete. Do not set it over small clumps of grass. Instead, use a shovel to clear a space, then tamp the plate down. When rotating the rod, maintain a constant weight on the plate so it is not disturbed. Do not place your feet near the handle of the plate.

Wooden stake

In sandy or marshy ground or the loosely packed soil sometimes encountered on highway and railroad embankments, use a wooden stake. The stake is made from a piece of 5-cm by 5-cm wood cut 60 to 90 cm long. To use the stake as a turning point, hammer it into the ground to a firm depth, leaving about 5 cm exposed. Then, drive a double-headed nail into the top until the bottom head rests against the wood. The top of the nail serves as the turning point.

Stability

During each setup, the leveling instrument and the rods may change elevation because of settlement or rebound caused by the type of ground cover. To minimize error from such movement, the backsight and foresight should be observed nearly simultaneously. This requires two rods and an efficient instrument and leveling unit.

Instrument movement can be reduced by proper use and maintenance of the tripod. Systematic error resulting from consistent movement of the instrument can be eliminated by using one of two observing procedures. In one procedure, known as micrometer leveling, two elevation differences are measured in two directions during each setup. If the instrument settles significantly during either or both measurements, the observations will not lie within a limit imposed on the difference between the results. Averaging the results practically eliminates any remaining systematic error.

In the other procedure, three-wire leveling, the first reading of the setup is made on the backsight of odd-numbered setuips, and on the foresight of even-numbered setups. Consistent settlement of the leveling instrument will cause the results of odd setups to be too large and the results of even setups to be too small by a similar amount. Thus, over a section with an even number of setups, the errors nearly cancel. Rod movement is minimized by placing the turning points so they provide sufficient stability. Systematic error caused by consistent movement during each setup can be limited somewhat by the procedures described previously. If the rods are allowed to rest on the points for 20 seconds before making an observation, any remaining movement should be negligible.

When proceeding from one setup to the next, the forward turning point must not move. This can only by assured by the rodman’s diligence and by comparing repeated levelings of the section. Averaging the results of the two runnings, made in opposite directions, nearly eliminates any systematic accumulation of error resulting from consistent movement.

Leveling Procedures

Pacing Balanced Setups

Many errors in leveling are reduced in magnitude by balancing setups. The distance from the leveling instrument to the foresight point must equal the distance from the instrument to the backsight within the tolerance for the survey. When a pacer is a member of the leveling crew, the pacer should lay out a balanced setup and set the foresight point before the rodman arrives from the previous setup. This permits leveling to proceed more efficiently than when rodmen set the turning points.

To lay out a balanced setup, count paces from the backsight point to the next position for the instrument and make a mark. Then, take an equal number of paces to the spot where the foresight point is to be set.

Paces need not be of any specific length: rather, they must be consistent. A beginning pacer (or rodman) should mark a 50-meter course on level ground and pace it several times, striving to develop consistency in the number of paces taken. Most individuals take paces about one meter in length. Therefore, to make a change of less than 5 m in the length of a setup, take the number of paces equivalent to the change.

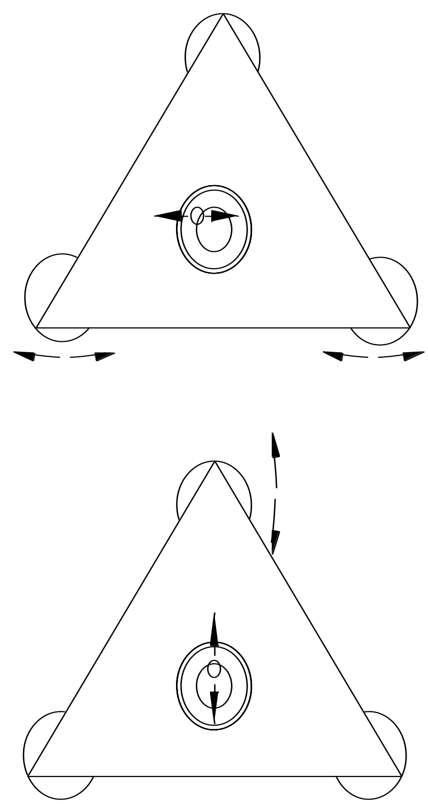
Some individuals find that they routinely pace “long” (foresight distance longer than backsight distance) or “short” (foresight distance shorter than backsight distance). Learn to compensate for such tendencies to prevent accumulating a large total of imbalances during a section of leveling. For this reason, do not move more than one setup ahead of the rest of the unit. If the observer or recorder alerts the pacer as soon as the total imbalance for the section exceeds the tolerance for one setup (see FGCS specifications at the end of this chapter), the pacer can reduce the total gradually instead of making one or two extremely unbalanced setups.

Instrument Operations

Leveling the instrument.

At every setup the instrument must be roughly leveled. A circular level (known also as a spot or bull’s eye) is attached to the body of the instrument for this purpose. The bubble in the level is centered to ensure that the compensator (or the bubble in the tubular level of a spirit-level instrument) is freely suspended.

To rough-level the instrument, first, turn it so the circular level and one foot screw form a line perpendicular to the line formed by the other two foot screws (Figure 5-21). Turn the two foot screws simultaneously, either toward or away from each other, to effect side-to-side movement of the bubble. Then, turn the first foot screw to move the bubble forward and backward, until it is centered.



Centering the Bubble in a Circular Level
Figure 5-21

Many instruments are equipped with mirrors or other optical mechanisms for observing the circular level. These mechanisms may invert or rotate the bubble image. For example, the bubble on the NI 002 may be observed either in a mirror or through the eyepiece. In the mirror image, side-to-side movement is opposite to, what the observer would expect when looking directly down on the bubble. The relationships of the eyepiece image to the direct view of the bubble is more complex, depending on the orientation of the eyepiece. If the eyepiece is turned to the side of the instrument, side-to-side movement appears forward-backward, and vice versa.

With practice, the observer can learn to level the instrument efficiently while observing the bubble through the eyepiece.

Circular level adjustment

At the start of work each day, and after any severe shock to the instrument, the observer should check that the circular level is properly adjusted. A circle is inscribed on the vial glass to provide a reference for assessing the bubble's movement. When the instrument is sufficiently level, the bubble should remain precisely centered as the instrument is slowly turned through a full circle about the vertical axis. If this is not the case, the level should be adjusted as follows:

1. Turn the instrument until the bubble displacement from center is at a maximum.

2. Using the tribrach foot screws, bring the bubble halfway back to center.

3. Using the three or four small screws supporting the circular level, adjust the bubble until it is centered. Each screw is adjusted by slipping an adjusting pin through the screw and turning it so as to maintain downward pressure on the base washer. Do not adjust by depressing one screw and retracting another. If the limit of downward motion is reached on the screw being turned, retract all the screws and start adjusting them again. Do not force the level vial against its base.

4. Rotate the instrument slowly through a full circle. If the center of the bubble does not stay within 2 mm of the center of the level vial, repeat the entire adjustment procedure.

Parallax adjustment

Before observing with the instrument, the observer should check for parallax. Parallax results when the focused image of an object does not lie on the same focal plane as the focused image of the reticle lines. To test for this, point the instrument at the sky or at a distant light-colored surface. Focus the ocular so the lines appear sharp and black, without straining the eye. Then, focus the objective on the scale of a leveling rod about 40 m (130 ft) away, and move the eye up and down, slowly, across the ocular. If the lines appear to move over the rod graduations, parallax is present. To eliminate the parallax, refocus the objective until no parallax is evident. If the image of the graduation is not distinct, focus the ocular by the small amount necessary to make it appear distinct.

Making an observation

After setting up and leveling the instrument, the observer should point it quickly at the appropriate leveling rod. With most instruments this is done by sighting along the top of the telescope. Once the instrument is roughly pointed, the rod is observed through the telescope and the tangent screw is turned to align the image of the rod scale with the reticle lines.

The reticle normally includes a single vertical line with three horizontal lines crossing it. The top and bottom stadia lines are used to measure sighting distance. The longer, middle line defines the horizontal line of sight. When leveling with scales having block graduations, a straight line is sufficient. The procedure is the same for bar-code levels. However, the operator must ensure that the instrument is accurately focused on the rod prior to observation.

Instrument maintenance

Before putting the instrument away each day gently wipe it with a clean soft cloth to remove dust and moisture. If the instrument has been exposed to rain or mist, it should also be allowed to stand at room temperature over night. If the instrument is dirty or greasy, or if it has been exposed to salt water, clean all nonglass surfaces with a cloth dampened in denatured alcohol. Dust the objective and ocular lightly with a lens brush. Then, if necessary, wipe the lenses with lens paper that has been moistened with a small amount of lens cleaning solution. Care must be taken not to scratch the glass surfaces.

The instrument, with the lens cap attached, should be stored securely in a specially designed padded case. To protect the instrument from jolts and vibration while the truck is in transit, the instrument should either be carried in a foam-lined box bolted to the truck bed, or strapped onto an empty seat in the truck. If the instrument is to be shipped to another location, it should be packed in its case and placed inside a well-padded carton, with an appropriate warning label affixed on the outside of the package.

At least once very 18 months, the instrument should be cleaned and adjusted on a collimator by a qualified technician.

Sighting Distance

The sighting distance between the instrument and a leveling rod is normally computed by the stadia method. To use the method, a full or half stadia interval must be computed from stadia readings made while observing. To balance the sighting distances of awkward setups quickly, before any observations are recorded, the stadia interval may also be determined by counting the number of rod units observed between the stadia lines (full interval). If a bar-code level is used, a quick-distance-only observation is all that is needed.

Determining the Stadia Factor

The stadia factor is set by the instrument manufacturer for converting the stadia interval to sighting distance. In modern instruments used for geodetic leveling the stadia factor is typically 100 or 333.3. The reticle is etched in glass to ensure that the factor does not change. The stadia factor need not be determined in the field when using such an instrument.

Collimation Check

Under field conditions the collimation error of a leveling instrument is measured by obtaining a set of observations called the collimation check ("C-shot" or "peg test"). Collimation error is limited by adjustment of the instrument. Because the adjustment can change easily under field conditions, thus changing the collimation error, make the collimation check at least once a day with most instruments. In addition, make the check any time that an instrument sustains a severe shock or seems to function abnormally.

The collimation check has two purposes: to prove that the instrument is properly adjusted within the standard of accuracy required for the survey, and to provide a collimation factor with which to correct data from unbalanced setups.

Two elevation differences are observed in the same direction, but with different setups, between the same two turning points. Each setup has an imbalance of sighting distances. Since the elevation differences measured between the same two points should be equal, any difference between them is the result of the effects of collimation error, pointing error, refraction, and curvature.

Leveling the section

The entire leveling crew must exercise the greatest possible attention to detail while leveling each section of the line. Each day check and maintain the instruments and equipment.

Explain specifically any deviation from the leveling routine. Record the following: (1) date, (2) time at which the leveling of each running began and ended, (3) personnel, and (4) equipment involved. Describe any symptoms of potential equipment failure at the time they occur.

Keep sighting distances within the prescribed tolerance for the order and class of the survey. If shimmer presents a challenge, shorten the sighting distance until acceptable readings can be made. In addition, do not allow the line of sight to pass closer than 0.5 m to the ground or to any intervening object. These precautions should reduce the error caused by refraction.

Balance the sighting distances as closely as possible on every setup, at least within the prescribed tolerance. This reduces the effects of collimation error, refraction, and curvature.

Keep the total of the setup imbalances as small as possible, at least within the prescribed tolerance. If the accumulated imbalance becomes large during the leveling of the section, adjust the remaining setups to diminish it gradually. Do not try to correct for it with a few extremely unbalanced setups.

Complete the section as efficiently as possible, with no breaks during the series of setups. Only properly described, permanent monuments may serve as beginning and ending points. If the section cannot be completed or if a blunder occurs, reject the data collected so far.

Except in the case of an incompleted section, do not reject any data in the field. If a blunder is found to have occurred after the fact, document it as clearly as possible on the recording form or by letter to the project office.

After completing one section, advance without a break to the first setup of the next section whenever possible. The recorder may complete the ending and beginning running records from the new setup.

Three-Wire Leveling

Until the advent of electronic bar code instruments or those equipped with micrometers, three-wire leveling was the most precise method for measuring elevation differences. However, without a micrometer, scale readings must be estimated and only one elevation difference can be measured efficiently during each setup. The method described here can provide differences with a precision sufficient for second- or lower-order surveys.

The instrument need only provide a single, consistent line of sight during each setup. Two nearly identical rods are used to permit nearly simultaneous observation of the backsight and foresight. A calibrated scale is necessary on each rod. To estimate readings with the greatest precision, the calibrated scale is graduated with blocks, preferably in a checkerboard pattern.

During each setup, six readings are made. The intercepts of the upper, middle and lower reticle lines ("wires") are read from the calibrated scale of rod 1. The three readings are estimated to the nearest tenth of a rod unit.

To check the internal consistency of the three precise readings, the half stadia intervals s_U and s_L, are computed and their difference is compared to a tolerance.

Second, three more intercepts are read and checked from rod 2. The sighting distances are computed by summing the half stadia intervals for each rod. Then, the imbalance is computed and checked against the tolerance.

Rod 1 is observed first during each setup. Thus, the backsights and foresights are observed in alternate order on alternate setups, to reduce systematic error. This is especially important when using a compensator instrument, to prevent the accumulation of systematic error due to consistent variation in the collimation error.

The elevation difference for each setup is the difference between the mean of the three backsight readings and the mean of the three foresight readings. It is not usually computed until the entire section is completed.

With three-wire leveling certain blunders cannot be detected. For example, a mathematical check to detect transposition of the foresight and backsight observations is not possible. Neither is a mathematical check to detect disturbance of the instrument between the backsight and foresight observations.

Instructions

The following instructions apply to a properly adjusted instrument without a micrometer (or with the micrometer locked in position), used with a pair of calibrated rods, each having one calibrated scale and a check scale.

1. EQUIPMENT RECORD: At the start of each day or at any change of the observer, equipment, or collimation factor, prepare the survey-equipment record. Include the date, the instrument serial number, the rod numbers, the local time zone and time, whether temperature is measured in Fahrenheit or centigrade units, the collimation factor at the last check, and the initials of the observer, recorder, and rodmen. CAUTION: Each work day check all serial numbers against the actual equipment used.

2. Establish a balanced setup with the backsight rod plumbed on the beginning control point of the section, the foresight rod plumbed on a turning point, and the instrument set in line between them. Check for parallax, using the instrument, check that the rods are plumb.

3. BEGINNING RUNNING RECORD: Immediately before the leveling begins, prepare the beginning running record. Include the following items of information: The survey-point serial number of the beginning control point, the stamping on the point (or designation if no stamping exists), a rubbing of the control point, the local time. CAUTION: Check that the backsight rod is, in fact, on the point identified. The recorded stamping must correspond to that on the mark leveled.

4. ROD 1: Level the circular level on the instrument while pointing at rod 1. With a spirit-level instrument, use the tilting screw to center the bubble precisely in the tubular level. Read the intercepts of the three reticle lines with the precise scale, top to bottom. Estimate each reading to tenths of a rod unit. Record in the backsight column.

5. READING CHECK: Compute and record the half-stadia intervals. The upper interval, s_U, is the difference between the upper and middle readings,

and the lower interval, s_L, is the difference between the middle and lower readings. Check that s_U - s_L is no more than 0.3 rod unit. If it exceeds this tolerance, reobserve rod 1, beginning at step 4.

6. If the tolerance is satisfied, compute the full-stadia interval, $s_U + s_L$, and record it. Compute the sighting distance and check that it is within the tolerance for the survey. For convenience, if recording on paper, convert the tolerance to a permissible stadia interval in rod units, and use this value to make the check. For example, if the rod units are centimeters, the stadia factor is 333, and the tolerance for sighting distance is 60 m, then the tolerance for the stadia interval is 18.0 cm [(60 m x 100 cm/m) ÷ 333 = 18.0 cm].

7. Compute the mean of the three readings. The mean may be computed quickly by examining the last digits of s_U and s_L. If s_U is three tenths larger than s_L, add 0.10 to the middle reading. If s_U is two tenths larger than s_L, add 0.07. If s_U is one tenth larger, add 0.03. If s_U equals s_L, the middle reading is the mean. Similarly, if s_U is three tenths smaller than s_L, subtract 0.10 from the middle reading. If s_U is two tenths smaller, subtract 0.07. If s_U is one tenth smaller, subtract 0.03. The mean always ends with the numeral “0”, “3”, or “7” in the hundredths column. Add the sum the three readings to the sum from the previous setup. Also, compute the sum of the means computed so far. CAUTION: Do not move the backsight rod and turning point until the data for the entire setup have been checked.

8. ROD 2: Point the instrument toward rod 2. Do not relevel the circular level. With a spirit-level instrument, recenter the bubble in the tubular level. Make three readings as before. Record in the foresight column.

9. Perform the reading check as in steps 6 through 8.

10. Compute and check the imbalance (backsight distance minus foresight distance). If it exceeds the tolerance for the survey, reobserve the setup, changing the position of the foresight rod or the instrument.

11. Check the accumulated imbalance for the section. If it is more than the tolerance for one setup, alert the rodmen so the next setup or setups can be adjusted accordingly.

12. NEXT SETUP: Advance to the next setup. Leave the former foresight rod (and turning point) in place to become the next backsight rod. Move the former backsight rod and turning point to the next foresight position. Set the instrument on a line halfway between the two rods. CAUTION: Do not remove the former foresight rod from its turning point. Keeping it centered on the point, pivot it to face the instrument.

13. Repeat steps 4 through 14.

14. As the unit approaches the end of the section, if required, adjust the sighting distances to make an even number of setups for the section. The final setup should have rod 1 on the control point and rod 2 on the last turning point.

15. After the last setup, compute and record the elevation difference for the section. Total the backsight readings and the foresight readings separately. Subtract the foresight total from the backsight total, and divide by 3. Similarly, total and subtract the backsight and foresight means. The two elevation differences should agree within the specified tolerance.

16. Compute and record the length of the section. Total the backsight and foresight stadia intervals, multiply the result by the stadia factor, and convert it into meters. A summary of the three-wire procedure is given in Figure 5-22.

During Each Setup

- ❑ Balance setup.
- ❑ Point instrument at rod 1.
- ❑ Level instrument and plumb rods.
- ❑ Read three reticle lines, top to bottom.
- ❑ Check half-stadia intervals and sighting distance.
- ❑ Repeat for rod 2.

Summary of Three-Wire Leveling Procedure
Figure 5-22

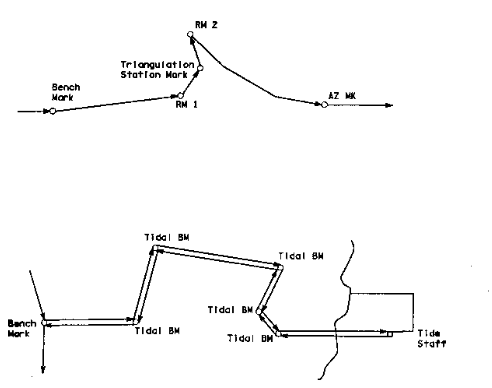
Precautions to Take at Control Points

The beginning and ending control points of a section are critical in three ways. First, the marks must be positively identified. Second, the control points must be clearly defined and properly used. Third, the stability of a mark that represents the end of a previous day's or week's leveling must be proved.

Identify the mark

Because of the proliferation of survey marks of all types and the possibility that control points may have been relocated without the knowledge of the leveling crew, each mark must be identified carefully. At the start of a section, identify only the beginning mark; identify the ending mark when the section is completed. When the rodman prepares the control point for the leveling rod, he or she should call out the stamping to the recorder. The recorder enters the stamping, as it is called out, and the corresponding survey-point serial number as it is given in the log.

Locations where marks may be easily misidentified include triangulation stations and tide or water-level stations. At these locations, marks are set in clusters, often only a few meters apart. To further complicate matters, many marks may be stamped with similar designations, perhaps bearing only one or two inconspicuous symbols to distinguish them. Level through such clusters with care. Avoid unnecessary spurs. For example, at a triangulation station, do not level separate spurs to the reference marks. Instead, level from one reference mark to the station, to the next reference mark, and then to the next control point in the line (Figure 5-23). Always accompany the level notes with a schematic of the routes performed.



Leveling Routes Through a Cluster of Control Points
Figure 5-23

Identify the control point

The leveling rod must rest on a clearly defined point. On a horizontal surface, the control point is the highest point of the mark. On a vertical surface, it is at the intersection of a pair of crossed lines (the exact center of a bench mark disk). The one exception to these rules occurs if some other point is specifically defined in the description of the control point. If any other point is used, describe it clearly in the leveling records.

Set a check point

The elevation of the control point at the end of a line segment leveled during one day must remain unchanged until leveling resumes the next day. To ensure this, set a check point wherever a day's leveling is not connected to previous leveling. The check point should be a solid point, clearly defined, and not on the same structure as the control point. Locate it no more than one setup away. Examples of check points include a marked point on a building foundation, or a marked point on a rock outcrop.

From a balanced setup between the control point and the check point, read the center wire of each rod. Write the readings in the "remarks" column of the recording form and compute the elevation difference, control point to check point. Also, write the difference on the field abstract for future reference.

When resuming work, observe a new difference between the control point and the check point, recording the difference as before. It should not differ from the first difference by more than 1 mm. If this check indicates movement has occurred, or that the rod was incorrectly placed on the control point, the original section leveled to the questionable point should be releveled in the opposite direction and closed.

If leveling is single run, set check points as follows: when leveling forward on the line, set one check point at the end of work each day; when leveling backward, set one check point at the start of work and set another at the end of work only if the previous day's leveling is not connected.

If leveling is double run, a check point need only be set at the end of a completed segment of line. It is practical to level the last section of the day in only one direction, setting no check point. The next day, level the section in the opposite direction, thus checking the stability of the end mark, as well as satisfying the double run requirement.

Leveling to Awkward Control Points

In the national network many existing control points are located where they cannot be easily leveled. These awkward points fall into two categories: (1) those mounted or etched on vertical surfaces, and (2) those mounted or etched on horizontal surfaces in positions requiring unusual equipment or procedures.

Locations where a control point may be found on a vertical surface include foundations and footings of large buildings and headwalls, and abutments of highway overpasses. The best way to level to a point in such a location is to intercept it directly. Another way is to use a short scale in lieu of the standard leveling rod.

Awkward locations for points on horizontal surfaces include the following: the top of a post or pedestal (too high to plumb the leveling rod properly and not large enough to support a rodman); a point less than 3 m (the length of a leveling rod) below an overhang or ceiling; and the underside of an overhang or eave. The best way to level to a point in the first location is to set the line of sight tangent to the highest point of the mark. At the second location, use a short rod. The third location requires that a rod be read while placed upside down against the point.

Releveling and Closing Sections

Often in single-run leveling, and always in double-run leveling, two or more runnings of a section are required. Just as two elevation differences observed on a single setup must be checked for agreement, the elevation differences obtained from multiple runnings of a section must be checked for agreement. The check is analogous to closing a loop by summing the differences of a series of setups beginning and ending on the same point and comparing the result to a tolerance. Thus, the check for agreement is called "closing the section."

If all leveling errors were eliminated, section runnings would agree exactly. However, since small amounts of random error are always present, tolerances have been set to ensure that runnings retained in the uncorrected leveling data disagree by no more than an amount consistent with the precision of the prescribed leveling procedure.

When multiple runnings of a section disagree by more than the tolerance, one or more blunders may have occurred. Blunders occur when prescribed procedures are not followed. The section must be releveled to obtain a sample of data, excluding blunders, that more accurately provides the elevation difference.

Releveling

If any of the following situations occurs during single-run leveling, relevel the section in the opposite direction and close the section. Releveling may be required by the project office for other reasons as well, after the data have been checked or suspected.

1. A procedural mistake is reported or suspected.
2. A tie check with previous leveling is not within tolerance.
3. One or more readings are missing or unclear, either in the computer or on a recording form.
4. The stamping recorded for a control point is missing or apparently incorrect, either in the computer or on a recording form.
5. One or more observations that do not meet specifications have been accepted because reading checks were computed improperly.

Closing the section

After a section is double run, check that the elevation difference from the two runnings does not exceed the maximum section misclosure for the order and class of the survey as specified in the FGSC specifications at the end of this chapter.

If the misclosure is within the tolerance, additional runnings are not required. If it exceeds the tolerance, relevel and recheck the section to satisfy two criteria: (1) All runnings likely to contain blunders (as determined by the rejection procedure) are rejected, and (2) at least one forward running and one backward running are accepted. A standard rejection procedure is used to reject statistically unreliable runnings.

When releveling to close the section, alternate the direction of leveling on each running, to maintain an equal number of forward and backward runnings. If systematic error persists in the leveling, the mean of the elevation differences may be biased in favor of the running direction that is in the majority. Equalizing the number of runnings in each direction prevents this bias.

Notify the project office if more than four runnings are required to close the section. To prevent excessive releveling, check that the leveling rods are being placed on the control points correctly and check that the points themselves are stable. Record any unusual features of the points in the leveling data. Check and adjust the rods and the instrument, especially the circular levels. On subsequent runnings, try shortening the sighting distances, balancing setups more precisely, varying the routes, and changing the duties of the crew members.

Notice that certain runnings, rejected when the number of runnings is three or four, may be accepted when the number increases. This is acceptable because the accuracy of the mean improves with a larger sample of data; it becomes easier to recognize if a running is different enough to be considered a blunder.

Field Records

Original records of field observations are the primary source of information for all future analysis and adjustment of a survey. As such, field records must accurately and completely present the results and conditions of leveling. The records may include both data recorded on forms and printouts of data recorded in computer memory.

Recording Observations

Leveling observations can be collected by using one of two methods. Observations may be (1) written on a recording form, or (2) keyed into computer memory and recorded on a magnetic tape or disk, or (3) automatically stored on a bar-code level and supplemented with a backup form. The computer-recording method is preferred to reduce errors and to simplify checking and archiving the data. Nevertheless, all three methods require that the recorder prepare clear, correct, and complete records.

When observations are written, the recording form serves both as a record and as a reminder of the checks and computations to be made. When observations are keyed, the backup recording form serves as a record to verify information that cannot be checked by the computer. Use standard recording forms.

Be sure to record correct identifying information. This cannot be overemphasized. Mistakes in any of the following entries will cause false elevation differences to be computed when leveling data are processed.

The recorded stamping, or designation of each control point, and its corresponding survey-point serial number determine the order in which the line is abstracted. This order must correspond to the way the line was actually leveled.

Finally, recorded descriptions of the equipment and the corresponding serial numbers determine which collimation and calibration corrections are to be applied. The make and model of rods and instruments should be specified by code if possible.

Write observations and remarks neatly in ink. Never recopy original records. If a mistake is made when computing, draw a straight line through the error and write the correct value above it. If a mistake is made when recording a rod reading, reobserve the setup and record the new readings on a new line. Crossed-out or illegible readings in written data are like readings missing from a computer-recorded tape: the entire section must be rejected.

When computing, round results to the appropriate number of decimal places. Elevation differences should usually be expressed in meters to five decimal places (four decimal places in three-wire leveling). Distances should be expressed in meters to one decimal place or in kilometers to two decimal places. Some calculators do not automatically round the result when displaying fewer decimal places than were computed. If using such a calculator, look at one extra decimal place and then round the result. If the number to be rounded ends with the numeral “5”, round to the nearest even number. For example, the collimation factor +0.0135 mm/m rounds to +0.014 mm/m, and the elevation difference -2.148145 m rounds to -2.14814 m.

Compute reading checks carefully. If they are computed incorrectly and one or more setups are accepted which do not meet the tolerance, the section must be releveled. This type of blunder can occur when preliminary information is stored incorrectly in a computer. The observer should check for such blunders as work progresses.

Use the “remarks” column (or computer comment lines) freely. Record any activity or event which may affect the quality of leveling, such as frequent failure of reading checks, unusual atmospheric conditions, or difficulties with the computer-recording equipment. Give a complete description of the situation; time at which the activity occurred, the setup number, what happened, and the way it was handled. Be specific. Include the names of personnel involved and the serial numbers of the affected equipment.

At the end of the day, the party chief should check and initial the recorder’s work. Be sure to check that the designations and survey-point serial numbers are correct, as well as the other identifying information.

P:HSM5

3.5 Geodetic Leveling

Geodetic leveling is a measurement system comprised of elevation differences observed between nearby rods. Geodetic leveling is used to extend vertical control.

Network Geometry

Order Class	First I	First II	Second I	Second II	Third
Bench mark spacing not more than (km)	3	3	3	3	3
Average bench mark spacing not more than (km)	1.6	1.6	1.6	3.0	3.0
Line length between network control points not more than (km)	300[a]	100[a]	50[a]	50[b]	25[b]
Minimum bench mark ties	6	6	4	4	4

[a] Electronic Digital/Bar-Code Leveling Systems, 25 km

[b] Electronic Digital/Bar-Code Leveling Systems, 10 km

As specified in above table, new surveys are required to tie to existing network bench marks at the beginning and end of the leveling line. These network bench marks must have an order (and class) equivalent to or better than the intended order (and class) of the new survey.

First-order surveys are required to perform valid check connections to a minimum of six bench marks, <u>three at each end</u>. All other surveys require a minimum of four valid check connections, <u>two at each end</u>.

A valid "check connection" means that the observed elevation difference agrees with the <u>published</u> adjusted elevation difference within the tolerance limit of the new survey. Checking the elevation difference between two bench marks located on the same structure, or so close together that both may have been affected by the same localized disturbance, is <u>not</u> considered a proper check.

In addition, the survey is required to connect to any network control points within 3 km of its path. However, if the survey is run parallel to existing control, then the following table specifies the maximum spacing of extra connections between the survey and the existing control.

When using Electronic Digital/Bar-Code Leveling Systems for area projects, there must be at least 4 contiguous loops and the loop size must not exceed 25 km. *(Note: This specification may be amended at a future date after sufficient data have been evaluated and it is proven that there are no significant uncorrected systematic errors remaining in Electronic Digital/Bar-Code Leveling Systems.)*

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*Since these specifications and procedures are considered interim, NGS' analyses of the data will be the final determination if the data meet the desired FGCS order and class standards. The procedures must be followed exactly as prescribed, unless written permission is obtained from the FGCS Methodology Work Group Chairman. These interim specifications and procedures are currently being reviewed by the FGCS Methodology Work Group.

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Surveys Run Parallel to Existing Control Network

Distance, survey to control network	Maximum spacing of extra connections (km)
less than 0.5 km	5
0.5 km to 2.0 km	10
2.0 km to 3.0 km	20

Instrumentation

Order Class	First I	First II	Second I	Second II	Third
Leveling instrument					
Minimum repeatability of line of sight	0.25"[3]	0.25"[3]	0.50"[4]	0.50"[4]	1.00"
Leveling rod construction	IDS[7]	IDS[7]	IDS[7] or ISS	ISS	Wood or Metal
Instrument and rod resolution (combined)					
Least count (mm)	0.1[5]	0.1[5]	0.5-1.0[5,6]	1.0[6]	1.0[6]

IDS -- Invar, double-scale
ISS -- Invar, single-scale

[3] For Electronic Digital/Bar-Code Leveling Systems, 0.40" and 0.01 mm.

[4] For Electronic Digital/Bar-Code Leveling Systems, 0.80" and 0.1 mm.

[5] If optical micrometer is used.

[6] 1.0 mm if 3-wire method; 0.5 mm if optical micrometer.

[7] For Electronic Digital/Bar-Code Leveling Systems, Invar, single-scale.

Leveling rods must be one piece. A turning point consisting of a steel turning pin with a driving cap should be utilized. If a steel pin cannot be driven, then a turning plate ("turtle") weighing at least 7 kg should be substituted. In situations allowing neither turning pins nor turning plates (sandy or marshy soils), a long wooden stake with a double-headed nail should be driven to a firm depth.

According to at least one manufacturer's specifications, the electronic digital leveling instrument should not be exposed to direct sunlight. The manufacturer recommends using an umbrella in bright sunlight.

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Calibration Procedures

Order Class	First I	First II	Second I	Second II	Third
Leveling instrument					
Maximum collimation error, single line of sight (mm/m)	0.05	0.05	0.05	0.05	0.10
Maximum collimation error, reversible compensator-type instruments, mean of two lines of sight (mm/m)	0.02	0.02	0.02	0.02	0.04
Time interval between collimation error determinations not longer than (days)					
Reversible compensator	7	7	7	7	7
Other types	1	1	1	1	7[1]
Maximum angular difference between two lines of sight, reversible compensator	40"	40"	40"	40"	60"
Leveling rod					
Minimum scale calibration standard	N[2]	N[2]	N[2]	M	M
Time interval between scale calibrations (yr)	3	3	--	--	--
Leveling rod bubble verticality maintained to within	10'	10'	10'	10'	10'

N -- U.S. National standard
M -- Manufacturer's standard

[1] For Electronic Digital/Bar-Code Systems, collimation error determinations are required at the beginning of each day (0.05 mm/m = 10 arc seconds). Collimation data must be recorded with the leveling data and the daily updated value must be used during the daily data capture.

[2] For Electronic Digital/Bar-Code Rods, until the U.S. National Standard Testing Procedure is implemented, manufacturer's scale calibration standard is acceptable, provided the data used during the calibration are furnished in digital format.

Compensator-type instruments should be checked for proper operation at least every 2 weeks of use. Rod calibration should be repeated whenever the rod is dropped or damaged in any way. Rod levels should be checked for proper alignment once a week. The manufacturer's calibration standard should, as a minimum, describe scale behavior with respect to temperature.

Order Class	First I	First II	Second I	Second II	Third
Minimal observation method	micrometer[1]	micrometer[1]	micrometer[1] or 3 wire	3-wire[1]	center wire[1]
Section running[7]	DR, DS, or MDS	DR, DS, or MDS	DR	DR	DR
Difference of forward and backward sight lengths never to exceed: per setup (m)	2	5	5	10	10
per section (m)	4	10	10	10	10
Maximum sight length (m)[2]	50	60	60	70	90
Minimum ground clearance of line of sight (m)	0.5	0.5	0.5	0.5	0.5
Even number of setups when not using leveling rods with detailed calibration	yes	yes	yes	yes	---
Determine temperature gradient for the vertical range of the line of sight at each setup	yes	yes	yes	---	---
Maximum section misclosure (mm)	$3\sqrt{D}$	$4\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Maximum loop misclosure (mm)	$4\sqrt{E}$	$5\sqrt{E}$	$6\sqrt{E}$	$8\sqrt{E}$	$12\sqrt{E}$
3-wire method					
Reading check (difference between top and bottom intervals) for one setup not to exceed (tenths of rod units)	---	---	2	2	3
Read rod 1 first in alternate setup method	---	---	yes	yes	yes
Micrometer single-difference method					
Reading check (difference between low and high scale) for one setup not to exceed (micrometer units)	---	---	3	4	5
Read rod 1 first in alternate setup method	---	---	yes	yes	yes

(continued)

Order Class	First I	First II	Second I	Second II	Third
Electronic Digital/Bar-Code method					
$\Delta h_1 - \Delta h_2$ for one setup not to exceed (mm) for MDS procedure	0.30	0.30	0.60	0.70	1.30
Use multiple reading option to obtain each observation - minimum number of readings[d]	3	3	3	3	3
Double-scale rods, DS procedure					
Low-high scale elevation difference for one setup not to exceed (mm)					
With reversible compensator	0.40	1.00	1.00	2.00	2.00
Other instrument types: Half-centimeter rods	0.25	0.30	0.60	0.70	1.30
Full-centimeter rods	0.30	0.30	0.60	0.70	1.30

DS -- Double Simultaneous procedure; see summary of observing sequences
MDS - Modified, Double Simultaneous procedure; see summary of observing sequences
DR -- Double-Run
SP -- SPur, must be less than 25 km, must be double-run
D --- shortest one-way length of section in km
E --- length of loop in km

[a] Electronic Digital/Bar-Code method permitted.

[b] For establishing a height of a new bench mark, double-run procedures must be used. Single-run methods can be used to relevel existing work provided the new work meets the allowable section misclosure.

[c] Maximum sight length permitted unless the manufacturer recommends a maximum sight length which is less.

[d] If the standard deviation exceeds 0.1 mm, continue making readings until it is less than 0.1 mm or repeat observation.

Double-run leveling may always be used, but single-run leveling procedures can only be used where it can be evaluated using published height values, i.e., the difference in published height values can be substituted for the backward running. DS and MDS procedures are recommended for all single-run leveling, but single-difference procedures are permitted.

Rods must be leap-frogged between setups (alternate setup method). The date, beginning and ending times, cloud coverage, air temperature (to the nearest degree), temperature scale, and average wind speed should be recorded for each section, plus any changes in the date, instrumentation, observer, or time zone.

When using the DS and MDS procedures, the instrument need not be off leveled/releveled between observing the high and low scales when using an instrument with a reversible compensator. The low-high scale difference tolerance for a reversible compensator is used only for the control of blunders.

Summary of Observing Sequences
(Required for first-order; optional for other orders)

<u>DS Procedures</u>
With double-scale rods,
the following observing
sequence should be used:

backsight, low-scale
backsight, stadia
foresight, low-scale
foresight, stadia
off-level/relevel or
 reverse compensator
foresight, high-scale
backsight, high-scale

<u>MDE Procedures</u>
With bar-coded scale rods,
the following observing
sequence should be used:

backsight
backsight distance, standard error
foresight
foresight, distance, standard error
off-level/relevel

foresight, standard error
backsight, standard error

Office Procedures

Order Class	First I	First II	Second I	Second II	Third
Section misclosures					
(backward and forward) Algebraic sum of all corrected section misclosures of a leveling line not to exceed (mm)	$3\sqrt{L}$	$4\sqrt{L}$	$6\sqrt{L}$	$8\sqrt{L}$	$12\sqrt{L}$
Section misclosure not to exceed (mm)	$3\sqrt{D}$	$4\sqrt{D}$	$6\sqrt{D}$	$8\sqrt{D}$	$12\sqrt{D}$
Loop misclosures					
Algebraic sum of all corrected misclosures not to exceed (mm)	$4\sqrt{E}$	$5\sqrt{E}$	$6\sqrt{E}$	$8\sqrt{E}$	$12\sqrt{E}$
Loop misclosure not to exceed (mm)	$4\sqrt{E}$	$5\sqrt{E}$	$6\sqrt{E}$	$8\sqrt{E}$	$12\sqrt{E}$

L -- shortest one-way length of leveling line in km
D -- shortest one-way length of section in km
E -- length of loop in km

The normalized residuals from a minimally constrained least squares adjustment will be checked for blunders. The observation weights will be checked by inspecting the post adjustment estimate of the variance of unit weight. Elevation difference standard errors computed by error propagation in a correctly weighted least squares adjustment will indicate the provisional accuracy classification. A survey variance factor ratio will be computed to check for systematic error. The least squares adjustment will use models that account for:

- gravity effect or orthometric correction
- rod scale errors
- rod (Invar) temperature
- refraction--need latitude and longitude accurate to at least 6" or (preferably) vertical temperature difference observations between 0.5 and 2.5 m above the ground
- earth tides and magnetic field
- collimation error*
- crustal motion

*For Electronic Digital/Bar-Code Leveling Systems, collimation data must be recorded with leveling data and updated value must be used during data capture.

FGCSVERT (ver. 4.0 7/15/94) E

6 Washington Coordinate System

Most WSDOT surveying is done assuming a plane surface. However, in long distance surveys, such as national control surveys, the curvature of the Earth must be calculated into the results. Also, WSDOT uses coordinates which must be related to a state-wide system.

The Washington Coordinate System is a system of plane coordinates for points on the Earth's surface based on the North American Datum of 1983 (NAD83/91) as determined by the National Geodetic Survey (NGS) of the U.S. Department of Commerce. This system is based on a description of the Earth as an equipotential ellipsoid with the state of Washington related to it by use of north and south zones (both Lambert projections) each having horizontal control stations with carefully established, published coordinates. (See Chapter 4, Horizontal Control.)

In Washington, the effect of the change from NAD27 to NAD83/91 was to shift and rotate geographic coordinates roughly three hundred feet to the southwest and to change to the metric system.

State Plane Coordinates in Washington State may have been recorded either as NAD27 (old system) or NAD83/91 (new system). There are four important differences to remember about these systems.

1. NAD27 coordinates are in feet, NAD83/91 are in meters.

2. NAD27 coordinates are based on Clarke's Spheroid of 1866, NAD83/91 are based on GRS80.

3. NAD 27 coordinates are given as X (easting) followed by Y (northing) whereas NAD 83/91 coodinates are given as N (northing) followed by E (easting).

4. NAD 27 bearings were reckoned from south whereas NAD 83/91 reckons from north.

Some of the more important reasons we use state plane coordinates are:

1. By state law NAD83/91 is the only legal system in this state for recording coordinates. (WAC 332-130 also see RCW 58.20.180)

2. A traverse run with state plane coordinates can be closed on another point with state plane coordinates without having to run a loop.

3. Point positions can be reset easily if monuments are destroyed.

4. In many cases there are control points with state plane coordinates in the work area.

5. State plane coordinates provide a common means to tie projects together.

6. They provide a highly accurate mapping data base for Geographic Information Systems (GIS).

It is now necessary to obtain coordinates of a given monument by one of the following methods:

1. Check with Geographic Services to see if NAD83/91 coordinates are available.

2. Check with the Washington State Department of Natural Resources to see if some public agency or private land surveyor has measured and recorded the NAD83/91 coordinates of the point.

3. Use Global Positioning System (GPS) measurements to determine the coordinates. (Consult Geographic Services.)

4. Run a traverse of the appropriate order from the closest monument that has coordinates.

Project Datum Coordinates

Ground distances and angles can be measured and projects can be layed out and constructed working with project datum. During location and construction of highway projects, it is much easier to work with project datum coordinates than with Washington State Plane Coordinates. The procedure is to convert the State Plane Coordinates of the initial control points to project datum coordinates, construct the project using project coordinates, and then, upon completion of the project, convert the coordinates back to Washington State Plane Coordinates for archiving and future use.

Warning: Caution must be exercised when showing project datum coordinates on a plat or any other document. It is absolutely necessary to provide a clear explanation of the coordinates, state the combined factor (CF), and add or subtract a constant of sufficient size to make it impossible to mistakenly believe the project datum coordinates to be State Plane Coordinates. WSDOT adds 100 000.000 meters to both the northing and the easting to avoid confusion.

Washington State Plane Coordinates are projected upward from the state plane grid to the project datum plane in a two step process.

First, state plane grid coordinates (initial points) are divided by a *scale factor* abbreviated **SF** to convert them to coordinates on the ellipsoid surface (the surface most closely representing the earth's surface). See Figure 6-1.

Secondly, these coordinates are divided by an *elevation factor* abbreviated **EF** (sometimes referred to as the sea level factor) to convert them to coordinates on the project datum plane (ground). See Figure 6-2. **EF** is a function of the mean radius of the earth **R** (6 372 000 m) (the ellipsoid), the distance from the ellipsoid to the geoid **N** (negative in Washington), and the elevation above mean sea level **H** (geoid). The *geoid* is the earth's equipotential surface of the earth's attraction and rotation, which, on the average, coincides with mean sea level.

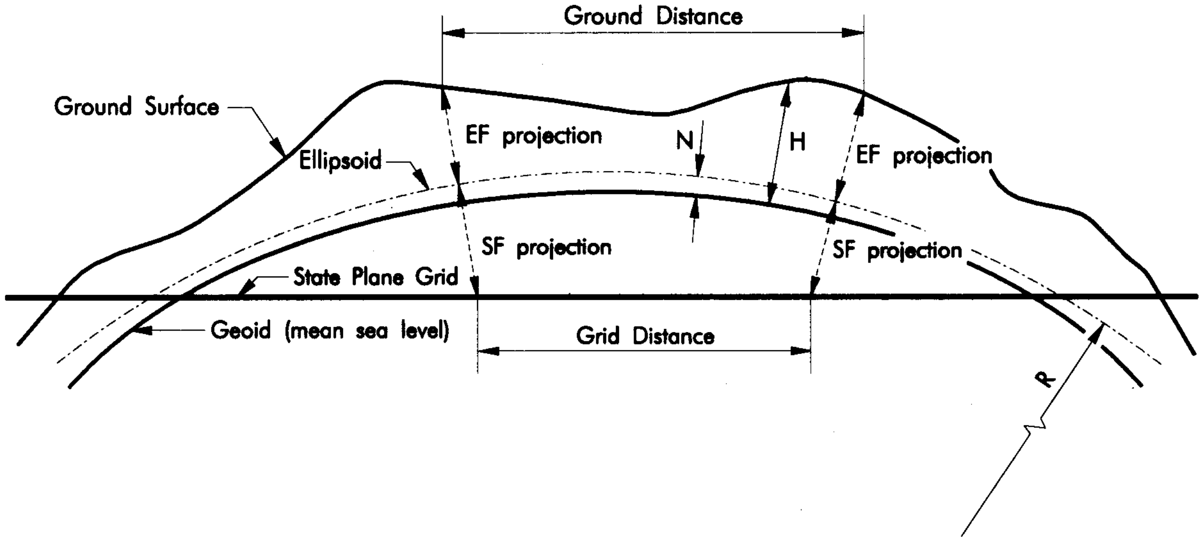
These two steps are usually combined by dividing Washington State Plane Coordinates by a *combined factor* abbreviated **CF** (scale factor times elevation factor)

Remember:

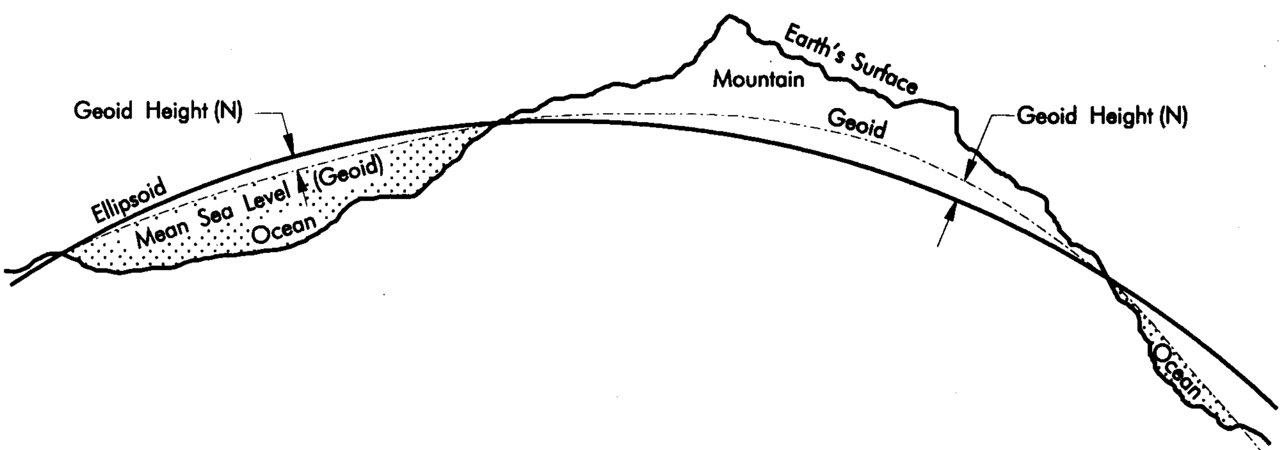
Divide Up	Grid Datum / CF = Project Datum
Multiply Down	Project Datum x CF = Grid Datum

For areas:

Divide Up	Grid Area / (CF) = Project (Ground) Area
Multiply Down	Project (Ground) Area x (CF) = Grid Area



Ground Distance and Grid Distance
Figure 6–1



Geoid — Ellipsoid Relationships
Figure 6–2

When Projection Factors are Needed

1. Project the initial Washington State Plane Coordinates to the ground surface. (See the procedure outlined below)

2. Traverse and set secondary control or photo targets using the project datum coordinates. (No scale factors or elevation factors used.)

3. Gather topography data. (No factors used.)

4. Design the project. (No factors used.)

5. Construct the project. (No factors used.)

6. Project the project datum coordinates to the Washington State Plane

Washington State Plane Coordinates to Project Datum (Manual Conversion Instructions)

The following is a step by step procedure for manually converting Washington State Plane Coordinates to project datum coordinates followed by computer instructions using the Engineering Systems Menu.

How to Make the Projection

You must document each step when projecting Washington State Plane Coordinates to project datum.

Note: all units must be in meters.

1. First, determine the maximum and minimum latitude of the project.

2. Calculate the mean latitude from the maximum and minimum latitudes.

3. Determine the zone (Figure 6.3) that the project is in, round the mean latitude to the nearest minute, and determine the scale factor **SF** by using the state plane projection tables (Figures 6.4 and 6.5).

4. For the elevation factor **EF**, take the mean radius of the earth **R**, 6,372,000 m, and divide it by the sum of the mean radius of the earth **R** plus the mean elevation from sea level for the project **H** plus the geoid height **N**.

   $$\mathbf{EF} = \mathbf{R} \;/\; (\mathbf{R+H+N})$$

   (Be sure to use all metric units and 8 significant digits). For the state of Washington, the geoid height **N** is usually around -20.0 meters.

5. Multiply the scale factor **SF** by elevation factor **EF** to obtain a combined factor **CF**. Again, use 8 significant digits.

   $$\mathbf{CF} = \mathbf{EF}(\mathbf{SF})$$

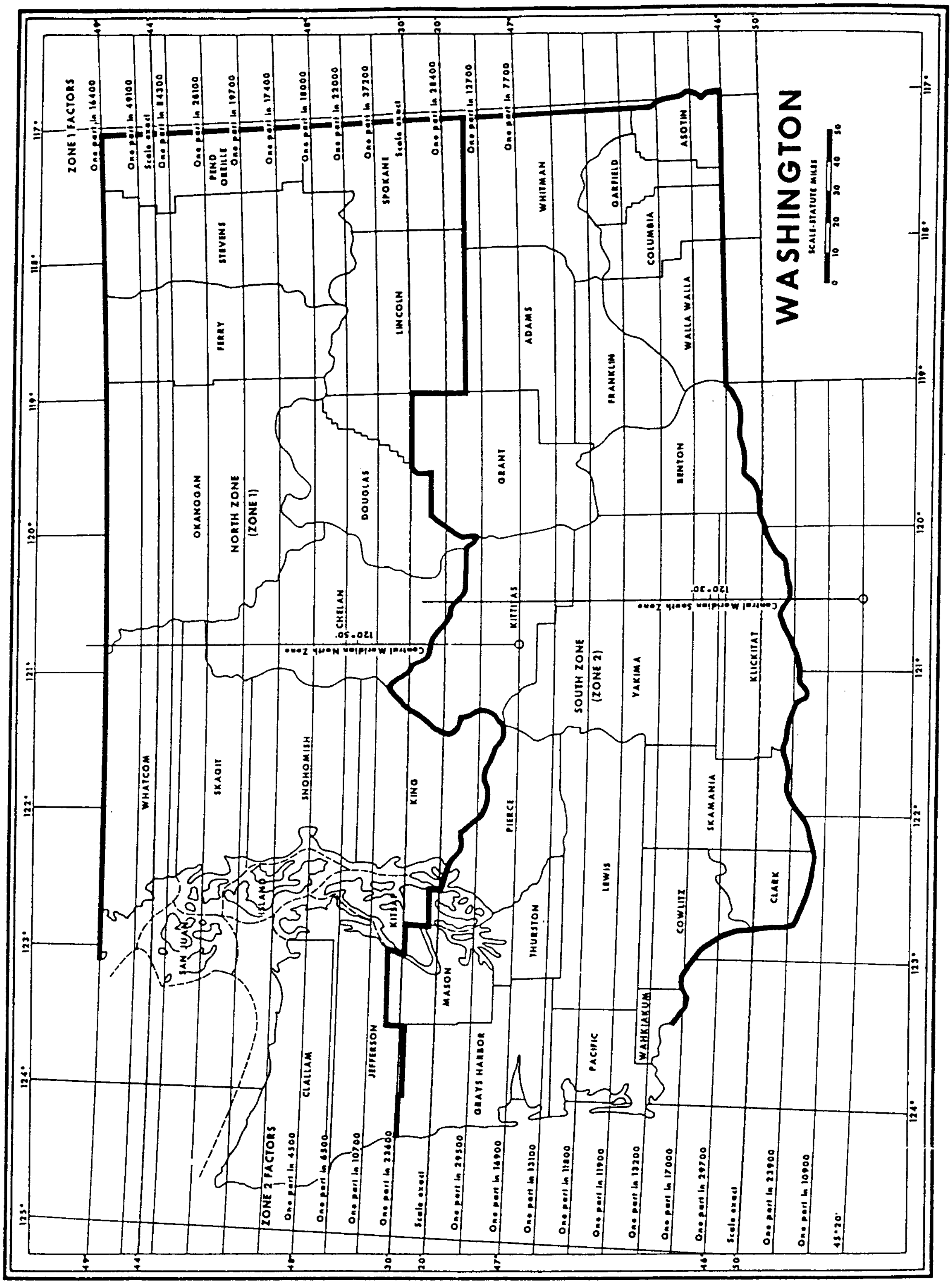
6. Take the coordinate values for the state plane northings and eastings and divide by the combined factor (**N/CF** and **E/CF**). To ensure that the project datum coordinates are not mistaken for State Plane Coordinates add 100 000.000 m to both the northing and the easting. This will provide a new set of coordinates at the ground surface.

7. To convert units, the U. S. Survey Foot must be used: **1 ft = (12 / 39.37) m**.

8. Each set of survey notes or plan sheets showing project datum coordinates must include the following note:

```
*************************************************************
*************************************************************
**** The values for the coordinates shown have been ***
*** projected to the project datum. To project back to ***
***** state plane coordinates, subtract 100 000.000 *****
******* meters then multiply by X.XXXXXXXX ******
*************************************************************
*************************************************************
```

At X.XXXXXXXX insert the combined factor **CF** using eight decimal places.



State Plane Coordinate Zones and Scale Factors
Figure 6–3

LATITUDE	SF	LATITUDE	SF	LATITUDE	SF	LATITUDE	SF
47° 01'	1.0001250	47° 36'	0.9999828	48° 11'	0.9999429	48° 46'	1.0000064
02'	1.0001196	37'	0.9999803	12'	0.9999433	47'	1.0000098
03'	1.0001142	38'	0.9999778	13'	0.9999437	48'	1.0000132
04'	1.0001089	39'	0.9999754	14'	0.9999443	49'	1.0000167
05'	1.0001037	40'	0.9999731	15'	0.9999449	50'	1.0000204
47° 06'	1.0000985	47° 41'	0.9999708	48° 16'	0.9999455	48° 51'	1.0000240
07'	1.0000935	42'	0.9999687	17'	0.9999464	52'	1.0000278
08'	1.0000885	43'	0.9999666	18'	0.9999473	53'	1.0000317
09'	1.0000836	44'	0.9999647	19'	0.9999483	54'	1.0000356
10'	1.0000788	45'	0.9999628	20'	0.9999493	55'	1.0000396
47° 11'	1.0000741	47° 46'	0.9999610	48° 21'	0.9999504	48° 56'	1.0000438
12'	1.0000694	47'	0.9999592	22'	0.9999517	57'	1.0000480
13'	1.0000649	48'	0.9999576	23'	0.9999530	58'	1.0000523
14'	1.0000604	49'	0.9999560	24'	0.9999544	59'	1.0000566
15'	1.0000560	50'	0.9999545	25'	0.9999558	49° 00'	1.0000611
47° 16'	1.0000517	47° 51'	0.9999532	48° 26'	0.9999573	49° 01'	1.0000656
17'	1.0000474	52'	0.9999518	27'	0.9999590	02'	1.0000703
18'	1.0000433	53'	0.9999506	28'	0.9999608	03'	1.0000750
19'	1.0000392	54'	0.9999495	29'	0.9999626	04'	1.0000798
20'	1.0000352	55'	0.9999484	30'	0.9999645	05'	1.0000847
47° 21'	1.0000313	47° 56'	0.9999474	48° 31'	0.9999665	49° 06'	1.0000897
22'	1.0000275	57'	0.9999465	32'	0.9999685	07'	1.0000947
23'	1.0000238	58'	0.9999457	33'	0.9999707	08'	1.0000999
24'	1.0000201	59'	0.9999450	34'	0.9999729	09'	1.0001051
25'	1.0000166	48° 00'	0.9999444	35'	0.9999752	10'	1.0001104
47° 26'	1.0000131	48° 01'	0.9999438	48° 36'	0.9999777	49° 11'	1.0001158
27'	1.0000097	02'	0.9999433	37'	0.9999802	12'	1.0001213
28'	1.0000064	03'	0.9999429	38'	0.9999827	13'	1.0001269
29'	1.0000032	04'	0.9999426	39'	0.9999854	14'	1.0001326
30'	1.0000000	05'	0.9999424	40'	0.9999882	15'	1.0001383
47° 31'	0.9999969	48° 06'	0.9999423	48° 41'	0.9999910	49° 16'	1.0001442
32'	0.9999939	07'	0.9999423	42'	0.9999939	17'	1.0001501
33'	0.9999910	08'	0.9999423	43'	0.9999969	18'	1.0001561
34'	0.9999882	09'	0.9999424	44'	1.0000000	19'	1.0001622
35'	0.9999855	10'	0.9999426	45'	1.0000032	20'	1.0001684

Scale Factors for North Zone
Figure 6–4

LATITUDE	SF	LATITUDE	SF	LATITUDE	SF	LATITUDE	SF
45° 21'	1.0001444	46° 06'	0.9999502	46° 51'	0.9999253	47° 36'	1.0000720
22'	1.0001382	07'	0.9999478	52'	0.9999266	37'	1.0000772
23'	1.0001322	08'	0.9999455	53'	0.9999281	38'	1.0000825
24'	1.0001262	09'	0.9999432	54'	0.9999297	39'	1.0000879
25'	1.0001203	10'	0.9999411	55'	0.9999313	40'	1.0000934
45° 26'	1.0001145	46° 11'	0.9999390	46° 56'	0.9999330	47° 41'	1.0000990
27'	1.0001088	12'	0.9999370	57'	0.9999348	42'	1.0001046
28'	1.0001032	13'	0.9999351	58'	0.9999367	43'	1.0001104
29'	1.0000976	14'	0.9999333	59'	0.9999387	44'	1.0001162
30'	1.0000921	15'	0.9999316	47° 00'	0.9999408	45'	1.0001221
45° 31'	1.0000867	46° 16'	0.9999300	47° 01'	0.9999429	47° 46'	1.0001281
32'	1.0000814	17'	0.9999284	02'	0.9999452	47'	1.0001342
33'	1.0000762	18'	0.9999269	03'	0.9999475	48'	1.0001404
34'	1.0000710	19'	0.9999255	04'	0.9999499	49'	1.0001467
35'	1.0000660	20'	0.9999242	05'	0.9999524	50'	1.0001530
45° 36'	1.0000610	46° 21'	0.9999230	47° 06'	0.9999550	47° 51'	1.0001595
37'	1.0000561	22'	0.9999218	07'	0.9999576	52'	1.0001660
38'	1.0000513	23'	0.9999207	08'	0.9999604	53'	1.0001726
39'	1.0000466	24'	0.9999198	09'	0.9999632	54'	1.0001793
40'	1.0000419	25'	0.9999189	10'	0.9999661	55'	1.0001860
45° 41'	1.0000373	46° 26'	0.9999181	47° 11'	0.9999691	47° 56'	1.0001929
42'	1.0000329	27'	0.9999174	12'	0.9999722	57'	1.0001999
43'	1.0000285	28'	0.9999167	13'	0.9999754	58'	1.0002069
44'	1.0000242	29'	0.9999162	14'	0.9999787	59'	1.0002140
45'	1.0000199	30'	0.9999157	15'	0.9999820	48° 00'	1.0002213
45° 46'	1.0000158	46° 31'	0.9999153	47° 16'	0.9999854	48° 01'	1.0002286
47'	1.0000117	32'	0.9999150	17'	0.9999889	02'	1.0002359
48'	1.0000077	33'	0.9999148	18'	0.9999925	03'	1.0002434
49'	1.0000038	34'	0.9999146	19'	0.9999962	04'	1.0002510
50'	1.0000000	35'	0.9999146	20'	1.0000000	05'	1.0002586
45° 51'	0.9999963	46° 36'	0.9999146	47° 21'	1.0000038	48° 06'	1.0002663
52'	0.9999926	37'	0.9999147	22'	1.0000078	07'	1.0002742
53'	0.9999890	38'	0.9999149	23'	1.0000118	08'	1.0002821
54'	0.9999855	39'	0.9999152	24'	1.0000159	09'	1.0002901
55'	0.9999821	40'	0.9999156	25'	1.0000201	10'	1.0002981
45° 56'	0.9999788	46° 41'	0.9999160	47° 26'	1.0000244		
57'	0.9999756	42'	0.9999166	27'	1.0000288		
58'	0.9999724	43'	0.9999172	28'	1.0000332		
59'	0.9999694	44'	0.9999179	29'	1.0000378		
46° 00'	0.9999664	45'	0.9999187	30'	1.0000424		
46° 01'	0.9999635	46° 46'	0.9999196	47° 31'	1.0000471		
02'	0.9999606	47'	0.9999206	32'	1.0000519		
03'	0.9999579	48'	0.9999216	33'	1.0000568		
04'	0.9999553	49'	0.9999227	34'	1.0000618		
05'	0.9999527	50'	0.9999239	35'	1.0000668		

Scale Factors for South Zone
Figure 6–5

WORK SHEET FOR CONVERTING STATE PLANE COORDINATES TO PROJECT DATUM

Project Name	**Zone** (North or South)	**Minimum Latitude**	**Maximum Latitude**	**Mean Latitude**	**Mean Elevation**	**Mean Geoid Height**

(SF)	**(EF)**	**(CF)**	**(N) and (E)** Project Datum	**Conversion to US Survey Foot**
Obtained by using the mean latitude from the table k value	$E.F.=\frac{R}{R+N+H}$ R = assumed radius of earth of 6,372,000 (meters) H = mean elevation from sea level for project (meters) N = mean Geoid height for project (meters)	$SF \times EF = CF$	$N = \frac{Y}{CF} + 100{,}000$ $E = \frac{X}{CF} + 100{,}000$	$N \times \frac{39.37}{12}$ $E \times \frac{39.37}{12}$

Point ID	**(Y) State Plane Northing** (meters)	**(X) State Plane Easting** (meters)	**(SF) Mean Scale factor** (8 significant digits)	**(EF) Mean Elevation factor** (8 significant digits)	**(CF) Combined Factor** (8 significant digits)	**(N) Project Datum Northing** (meters)	**(E) Project Datum Easting** (meters)	**Project Datum Northing** (feet)	**Project Datum Easting** (feet)

Calculated By:

Checked By:

Washington State Plane Coordinates to Project Datum (Computer Instructions)

This program will convert Washington State Plane Coordinates to project datum coordinates. This will generate a set of project datum coordinates that can be used to design and construct a project that has the scale and elevation factors for conversion already applied.

Note: Project datum coordinates have 100 000.000 meters added to the northing and easting values to prevent confusion with State Plane Coordinates.

Select the Survey System Menu from the Engineering Systems Menu.

SURVEY SYSTEM MENU
last revision - XXX XX XXXX

1: Set Up Program Parameters.
2: Process Collector File.
3: View 80 Column XSEC File with XVIEW.
4: Print Listing of XSEC File.
5: Plot a XSEC File.
6: Convert CGP XYZ File To/From Metric/English
7. Wash. State Plane to Project Datum.
8. Project Datum to Wash. State Plane.
9. Quit.

Do you want to run this program in interactive mode?

Yes No

PROGRAM MODE

Select **YES** if you want to enter the coordinate values individually.

Select **NO** if you have an “input file” of several points that will then be processed in the “Batch Mode”.

First we will illustrate the **“Interactive Mode”**.

---1.0 ------------------ Plane Coordinate to Project Datum -------------------

FILE INFORMATION FOR INTERACTIVE MODE

Output File Name ==> **C:\sdrdata\fname.sdr** REQ
(F1 to select) (default file extension (.SDR))

Output File Format ==> [**SDR/**CGP XYZ] REQ
(F2 to toggle)

Output Data Unit ==> [Meter/**Feet]** REQ
(F3 to toggle)

OK CANCEL

OUTPUT FILE NAME

Enter the directory, file name, and optional file extension for where the output file is to reside.

OUTPUT FILE FORMATS

SDR Format is the standard SDR format which displays 08KI records as Point Number, Northing, Easting, Elevation, and Class Code. 13KI records are also displayed in the file as Notes. Files in this format may be loaded to data collectors via Sokkia software.

CGP XYZ Format displays data by Point Number, Easting., Northing, Elevation, and Class Code. Files in this format can be processed by the Geographic Information Services Department and sent back to the regions for loading into various computer files.

OUTPUT DATA UNITS

Select the output data units, either feet or meters . (Note: The calculation results displayed at the end of this program will be in both feet and meters.)

After selecting OK the following panel displays.

---1.0 ------ Washington State Plane Coordinate to Project Datum ---------

Project Name ==> **SR 1 MP 100.0 to MP 104.0** REQ

PLANE COORDINATE DATA (IN METERS)

Zone ==> [**North/**South] REQ
Latitude NAD 83
Min => **047-00-00** REQ
Max => **047-04-20** REQ

Elevation NAVD 88 REQ
Min => **100** meters
Max => **200** meters
Geoid Height meters REQ
Min => **-20.5** meters
Max => **-20.1** meters

OK CANCEL

PROJECT NAME

Enter a project name up to 50 characters. This entry must not be left blank.

ZONE

Select the Zone within which your project lies. If part of your project is in the zone overlap area, be sure to choose the same zone for your entire project.

MINIMUM LATITUDE

Enter the minimum latitude for the project in Degrees Minutes Seconds. This can be scaled from a “Quad” map.

MAXIMUM LATITUDE

Enter the maximum latitude for the project in Degrees Minutes Seconds. This can be scaled from a “Quad” map.

The possible values for NAD 83 (North American Datum 83) latitude are:

NORTH ZONE: 47 deg. 00 min. to 49 deg. 20 min.

SOUTH ZONE: 45 deg. 20 min. to 48 deg. 10 min.

Note: If a project spans more than 6 minutes of Latitude, you may want to consider dividing it into more than one Project Datum.

ELEVATION

Enter the project minimum and maximum National Geodetic Vertical Datum (NAVD) 1988 elevations in meters.

GEOID HEIGHT

The Geoid Height is the seperation between the ellipsoid and the geoid. The geoid being the earth viewed as a hypothetical ellipsoid with the surface represented as mean sea level This number is negative in Washington State ranging in values from -23.0 to -17.0. This number can be obtained from the Washington State Plane Data sheet showing the coordinates for your project or from our Geographic Services Department.

```
----1.0 --- Washington State Plane Coordinate to Project Datum-----

POINT DATA
                                Point ID   ===>99              REQ
 Y State Plane Northing   (NAD 83) ===>134627.364      REQ
  X State Plane Easting    (NAD 83) ===>388301.004      REQ

                                   OK            CANCEL
```

Note: Northing and Easting coordinate values must be in Meters.

POINT ID

Enter a number from 1 to 9999

Y STATE PLANE NORTHING

Enter State Plane Northing (in meters)

X STATE PLANE EASTING

Enter State Plane Easting (in meters)

Enter OK to run the program.

The following panel is then displayed.

---1.0 --- Washington State Plane Coordinate to Project Datum----

CALCULATION RESULTS

Project Name	:	**SR 1 MP 100.0 to MP 104.0**			
Zone	:	**North**			
Point ID	:	**99**			
Max. Latitude	:	**47 - 04 - 20.00000**			
Min. Latitude	:	**47 - 00 - 00.00000**			
Mean Latitude	:	**47 - 02 - 10.00000**			
Mean Elevation	:	**150.000**	meters		
Mean Geoid Height	:	**-20.300**	meters		
State Plane Northing	:	**134627.364**	meters		
State Plane Easting	:	**388301.004**	meters		
Mean Scale Factor	:	**1.00011874**			
Mean Elevation Factor	:	**0.99997965**			
Combined Factor	:	**1.00009838**			
Project Datum Northing	:	**234614.121**	**m.**	**769729.828**	**ft.**
Project Datum Easting	:	**488262.807**	**m.**	**1601908.893**	**ft.**

ALL DONE CONTINUE CANCEL

Selecting **Continue** allows the user to enter another point.

Selecting **Cancel** will immediately terminate the program without any output.

Selecting **All Done** will finish the program and the following panel will be displayed.

********* SUCCESSFUL COMPLETION *********

The WSPC2PD has successfully completed processing.
The resulting Project Datum file is located at :

C:\sdrdata\filename.sdr

Also, a report file has been produced and it is located at :

C:\sdrdata\filename.RPT

At this point, the program will ask you if you want a printout of the report file.

Press **<ENTER>** to continue the printout process.

The report file must be printed and a copy of the Survey included in the "Final Records" of the project.

The following is an example of a **.RPT** file residing in the **C:\sdrdata*.RPT** directory.

```
                    Washington State Department of Transportation
                       WS Plane Coordinate to Project Datum
                        Project : SR1 MP 100.0 to MP 104.0
                    Date : 12/05/94    Time : 06:45:08    Page : 1

=================================================================
--------------------PLANE COORDINATE DATA AND FACTORS--------------------
    Zone                    : North
    Max. Latitude           : 47 - 04 - 20.00000
    Min. Latitude           : 47 - 00 - 00.00000
    Mean Latitude           : 47 - 02 - 10.00000
    Mean Elevation          :  150.000
    Mean Geoid Height       :  - 20.300
    Mean Scale Factor       :     1.00011874
    Mean Elevation Factor   :     0.99997965
    Mean Combined Factor    :     1.00009838

        ------------------------------ RESULTS FOR POINT ID  99 ------------------------------
    State Plane Northing     :134627.363  meters
    State Plane Easting      :388301.004  meters
    Project Datum Northing   :234614.120  meters   769729.828  feet
    Project Datum Easting    :488262.807  meters 1601908.893  feet


    ************************************************************
    ************************************************************
    **  The values for the coordinate shown have been projected    **
    **  to a project datum.  To project back to state plane        **
    **  coordinates,  subtract 100 000.000 meters then multiply   **
    **  by     1.00009838                                          **
    ************************************************************
    ************************************************************

    Notes:
```

Now we will illustrate running the program in “**Batch Mode”.**

WS-Plane Coordinates to Project Datum

Do you want to run this program in
interactive mode ?

Yes **No**

PROGRAM MODE

Select **No** to run the program in “Batch Mode”.

The following then displays.

--- 1.0 -------------- Plane Coordinate to Project Datum------------------

FILE INFORMATION FOR **BATCH MODE**

Input File Name (F1 to select)	==> **C:\sdrdata\fname.SDR** (default file extension (.SDR))	REQ
Input File Format (F2 to toggle)	==> [SDR/**CGP XYZ]**	REQ
Output File Name (F3 to select)	==> **C:\sdrdata\fname.SDR** (default file extension (.SDR)	REQ
Output File Format (F4 to toggle)	==> [**SDR/**CGP XYZ]	REQ
Output Data Unit (F5 to toggle)	==> [Meter/**Feet]**	REQ

OK CANCEL

INPUT FILE NAME

This file may be a file generated in SDRMAP or a file from Geographic Information Services in the CGP XYZ format.

INPUT FILE FORMAT

Select the type of format your input file is: SDR or CGP XYZ. If the user selects a file format that is different than the actual file format, then the program will not produce the correct results.

OUTPUT FILE NAME

Enter the directory, file name, and optional file extension of where the Output file is to reside.

OUTPUT FILE FORMATS

SDR Format is the standard SDR format which displays 08KI records as Point Number, Northing, Easting, Elevation, and Class Code. 13KI records are also displayed in the file as Notes. Files in this format may be loaded to data collectors via Sokkia software.

CGP XYZ Format displays data by Point Number, Easting., Northing, Elevation, and Class Code. Files in this format can be processed by the Geographic Information Services Department and sent back to the regions for loading into various computer files.

OUTPUT DATA UNITS

If the SDR File Format has been selected, the Output Data Units selected must be Feet. The calculation results displayed at the end of this program will be in both feet and meters.

After selecting **OK** the following panel displays. This is the same panel that displays when running interactive mode with the same data input requirements as noted previously.

```
---1.0 ------ Washington State Plane Coordinate to Project Datum ---------

     Project Name ==> SR 1  MP 100.0 to MP 104.0                REQ
   -----------------------------------------------------------------------------
PLANE COORDINATE DATA (IN METERS)

     Zone ==> [North/South]                                     REQ
     Latitude NAD 83
      Min => 047-00-00                                          REQ
      Max => 047-04-20                                          REQ

     Elevation NAVD 88                                          REQ
      Min =>     100    meters
      Max =>     200    meters
     Geoid Height       meters                                  REQ
      Min =>    -20.5   meters
      Max =>    -20.1   meters
                            OK            CANCEL
```

Selecting **OK** will result in the following panel being displayed when the input file has been successfully converted.

********* SUCCESSFUL COMPLETION *********

The WSPC2PD has successfully completed processing.
The resulting Project Datum file is located at :

C:\sdrdata\filename.sdr

Also, a report file has been produced and it is located at :

C:\sdrdata\filename.RPT

At this point, the program will ask you if you want a printout of the report file. Press **<ENTER>** to continue the printout process.

The report file must be printed and a copy of the Survey included in the “Final Records” of the project.

An example of an .RPT file is shown in the interactive mode section of this documentation.

Project Datum to Washington State Plane Coordinates (Computer Instructions)

This Program, PD2WSPC (Project Datum to Washington State Plane Coordinates), converts Project Datum coordinates to Washington State Plane Coordinates. It is designed to work with the previous program WSPC2PD (Washington State Plane Coordinates to Project Datum). It will globally apply a "combined factor", and subtract 100,000 meters from the Northing and Easting coordinates.

Select the Survey System Menu option from the Engineering Systems Menu.

```
SURVEY SYSTEM MENU
last revision - XXX XX XXXX

1:  Set Up Program Parameters.
2:  Process Collector File.
3:  View 80 Column XSEC File with XVIEW.
4:  Print Listing of XSEC File.
5:  Plot a XSEC File.
6:  Convert CGP XYZ File To/From English/Metric
7:  Wash. State Plane to Project Datum.
8:  Project Datum to Wash. State Plane.
9.  Quit.
```

Select option (8) from the Survey Systems Menu and the following panel displays.

```
1.0           Project Datum to WS Plane Coordinate

COMBINED FACTOR INFORMATION

Combined Factor   ==>1.00009838                          REQ
FILE INFORMATION
Input File Name   ==>  C:\sdrdata\fname                  REQ
     (F1 to select)    ( default file extension : (.SDR) )

Input File Format  ==>  [CGP XYZ/SDR]     (F2 to toggle)   REQ
Input Data Unit   ==>  [Meter/Feet]       (F3 to toggle)   REQ

Output File Name   ==>  C:\sdrdata\fname                 REQ
     (F4 to select)    ( default file extension : (.SDR) )

Output File Format ==>  [CGP XYZ/SDR]     (F5 to toggle)  REQ
Output Data Unit   ==>  [Meter/Feet]      (F6 to toggle)  REQ

                    OK              CANCEL
```

COMBINED FACTOR

The Combined Factor is the (Scale Factor × Elevation Factor). This value may be obtained from the .RPT file that is generated when executing the WSPC2PD (Washington State Plane Coordinates to Project Datum) program or calculated manually. If calculating the Combined Factor manually, follow the procedures shown in the “Worksheet for Converting State Plane Coordinates to Project Datum”.

INPUT FILE NAME

Enter the Input File name and extension.

INPUT DATA UNITS

Select Meters or Feet depending upon the units of your Input File.

OUTPUT FILE NAME

Enter the directory, file name, and optional file extension for where the Output file is to reside.

OUTPUT FILE FORMAT

SDR format is the standard SDR format which displays 08KI records as Point Number, Northing, Easting, Elevation, and Class Code.

CGP XYZ format displays data by Point Number, Easting, Northing, Elevation and Class Code.

OUTPUT DATA UNITS

Select the output data units, either feet or meters. Note: The caluculation results displayed at the end of this program will be in both feet and meters.

Now execute the program by selecting OK.

Note: Check the coordinates generated in the PD2WSPC (Project Datum to Washington State Plane Coordinates) program with your original file coordinates used in the WSPC2PD.(Washington State Plane Coordinates to Project Datum).

Caution: Spot check coordinates throughout the file.

P:HSM6

7

Photogrammetry

Photogrammetric surveys are used as a tool in reconnaissance, location design, PS&E, and construction projects. They are used for mapping, digital terrain modeling, cross sectioning, and alignment determination. All data collected is stored in digital form for computer manipulation. If a project will take more than four field crew days or there is a safety problem, using photogrammetry may be to your advantage.

The quality of a photogrammetric project depends on the quality of the aerial photos and the accuracy and placement of the field control.

If you are contemplating a project, contact the Photogrammetry Section early to assure optimum scheduling.

The telephone number is (360) 709-5540;
public 586-0288.

Glossary

B/W: Black and white photography, image defined in 26 shades of gray.

COLOR: Positive color or prints showing natural colors "NEGATIVE

COLOR FILM: Used for making positive color transparencies and either black and white enlargements and contact prints.

CONTACT PRINTS: Prints made from a negative or a diapositive in direct contact with sensitized material.

CONTROL POINT: Any station in a horizontal and/or vertical control system that is identified on a photograph and used for correlating the data shown on the photograph.

CROSS SECTION: Cross section profiles are measured in stereo plotters at right angles to and on stations as requested by the district. Computer listing quantities and line plots (plotted cross sections) are furnished to the district.

DIAPOSITIVE: A positive photographic print on a transparent medium. In photogrammetry, the term is used to refer to a transparent positive film used in a plotting instrument.

DTM: A digital terrain model is a numerical representation of the surface of the earth based on a set of x-y-z coordinates.

INFRARED: Films sensitive to longer (warmer) light wave lengths.

"INFRARED BLACK AND WHITE" for haze penetration, definition of water and moisture. "INFRARED FALSE COLOR," unnatural colors helpful in interpreting soils structure, moisture, diseased vegetation, etc.

LENS 152 mm: Wide angle lens with 152 mm focal length and 230 x 230 mm frame size. Our standard all-purpose camera.

LENS 304 mm: Narrow angle lens with 304 mm focal length and 230 x 230 mm frame size. Best for mosaics (where ground is not flat) and exhibit. Not suitable for mapping.

MODEL: The overlapping area of two photographs.

MOSAIC: An assemblage of overlapping photographs whose edges have been matched to form a continuous photographic representation of a portion of the earth's surface.

OBLIQUE: Lens is tilted obliquely to a point on the ground.

ORTHOPHOTOGRAPH: A photographic copy prepared from a perspective photograph in which the displacements of images due to camera tilt and ground relief have been removed.

ORTHOPHOTOMAP: A photomap made from an assembly of orthophotographs.

PHOTOGRAMMETRIC SERVICES: Plotting graphic data in map form or measuring and recording data in cross section or plane coordinate forms through the use of stereo plotting instruments.

PHOTOGRAMMETRY: The science or art of obtaining reliable measurements by means of photography.

PLANIMETRIC MAPPING: Includes all culture and natural features that can be defined in horizontal position, e.g., monuments, building, roads, railroads, streams, lakes, etc.

PLANNING ASSISTANCE: Photogrammetry liaison person will meet engineer in office and/or on project to help plan distribution and placement of control targets, etc.

PRE-MARK: A target placed on the ground used to identify a field control point.

STEREO-PLOTTERS: An instrument used by the photogrammetrist to view photographs in three dimensions in the production of maps and cross sections.

TOPOGRAPHIC MAPPING: Combines all planimetric features, plus contours and spot elevations.

VERTICAL: Lens points straight down. Photo looks like a map.

WING POINT: A ground control point located away from the center of the aerial flight line. Generally requires only vertical control.

Mapping

When mapping is required, photogrammetric surveys are quickly, safely, and economically made and should be requested whenever an effective savings in time and/or manpower can be realized.

To accomplish the comparative analysis of alternate routes, the determination of the correct map to order will be dictated by the character of the terrain being studied and the land use characteristics.

Todays technology of using Design Engineering work stations to solve engineering problems has completely changed photogrammetric techniques.

Features are recorded in three dimensions (x, y, and z coordinates) and topography is described by using breaklines (to describe a change of slope) and random points. These points, together with selected planimetric features, are used to produce contours or other engineering products.

When ordering a photogrammetric map, be sure to describe its intended use.

The order form provides a list of features to be collected in the digital terrain model (DTM). You can customize your map by requesting only some of the items, or have the photogrammetrist collect all visible features on the photography.

Choose the photogrammetric product that will best answer your needs. A **topographic** map contains man-made features, vegetation, hydrography, and contours. A **planimetric** map contains man-made features, vegetation, and hydrography, but no contours. The next step is to decide on the class of map. See Figure 7-1.

Reconnaissance

Reconnaissance mapping should be used where, in the opinion of the engineer, adequate information is unavailable and lack of current data may affect the proper development of a reconnaissance survey.

Request a scale of 1:5000 with a 3 m contour interval in areas of sparse land use, where the character of the topography is mountainous, where heavily timbered areas restrict more accurate photogrammetric determination of topography, or where it is known in advance that developed areas will not be appreciably altered by proposed routes.

Request a scale of 1:2000 with a 2 m contour interval in areas of moderate to intense land use, especially in urban areas. Consider this scale where the topography is slightly rolling or nearly level, and the requirements of a 2 m contour interval and more accurate delineation of detail would be necessary.

While existing geodetic control is used, additional ground control is usually necessary.

Location Design

Location design mapping is performed to furnish the engineer sufficient data to produce the best location for the highway and to aid in the preparation of detailed right of way plans. The aerial photography obtained at this stage will provide an invaluable record of conditions prior to construction.

In accomplishing design mapping, detailed and accurate surveys are made of sufficient width to ascertain all of the important factors which may control the position, physical characteristics, and geometric design of the highway route. The type and scale of mapping required in design phase will be governed by the amount of data necessary to fulfill the engineering requirements.

Generally, request a map scale of 1:1000 with a 1 m contour interval in flat areas or 1:1000 with a 2 m contour interval in rugged areas.

In urban areas of intense land use, request a map scale of 1:500 with a 0.5 m contour interval. When topographic design mapping is specified and the map sheets are also to be used as right of way or construction plans, the CADD system will allow the user to eliminate details not needed on a particular plan sheet or exhibit.

For bridge site maps, refer to the *Design Manual* Chapter 1110 for the appropriate map scale. For special design problems which may be encountered on bridge sites or interchange areas, request mapping at map scales of 1:200 to 1:400 with a $0.3\overline{3}$ m contour interval.

Horizontal and vertical mapping accuracies as related to photo scale are charted later in this chapter. Note that the accuracy is directly proportional to the scale of the photograph. In areas where the ground is obscured by timber or other dense vegetation, design mapping cannot ordinarily be accomplished to the degree of accuracy required. Approximate ground form lines can be indicated by dashed contours, but if a more accurate delineation of the topography is required, electronic survey methods should be used.

If preliminary cross sections are desired and there is a safety problem; a center line may be developed photogrammetrically from the existing painted stripes.

PS&E

In order to data accurate enough to be used for construction quantities, a photo scale of 1:3000 must be used.

Photogrammetrically generated data are especially beneficial in areas of rough terrain or where heavy traffic could be a problem for field crews. However, some field work may be necessary for data points not visible in the photos. In areas of heavy vegetation, photography should be obtained after clearing and grubbing.

Aerial photos can be used to update plans for existing conditions (as builts), for pit sites, interchanges, road approaches, drainage systems, etc. The photography taken at this stage will be particularly valuable if questions or disputes should arise after construction because it will show the conditions prior to any construction work.

For projects that require intermediate measurements or a final pay quantity, locate the survey control points so a minimum of maintenance is required.

To obtain the most accurate cross sections use black and white photography. This may necessitate a second flight in color for illustrative use.

While the digital terrain model will be used primarily for road design, contours may also be generated. These contour maps do have some limitations but they are very accurate and could be useful.

Photography

Aerial photography (stereo photos) is the basic source of information for photogrammetric surveys. Survey information is obtained through interpretation and analysis of stereo photography using photogrammetric plotters to make measurements and delineations.

Aerial photography and those products it produces also have extensive use as a visual communication tool for planning, property acquisition, engineering, construction, litigation, and public relations.

Aerial Photograph Section

The Aerial Photography Section consists of the photography unit which acquires high resolution data on the film negative. The photo lab unit uses these film negatives to produce hard copy (usually photo print positives on paper or film) in black and white, true color, or color infrared.

Aerial Photography

The aerial camera is used in two configurations in flight allowing vertical or oblique views to be taken of an area or subject. Photography taken with the high resolution (152 mm) mapping lens results in a metric quality negative suitable for use in the analytical stereo plotters when map compilation is required. It is also used for high altitude, small-scale negatives to cover a maximum area with each photo taken. Photography taken with a 305 mm reconnaissance lens is used for high quality or ultra large scale negatives. Mapping aerial photography can be taken with negative scales between 1:1200 and 1:64000, and oblique photography from 300 m to 10,000 m above sea level.

Negative Film Printing

Contact printing of black and white, true color, and color infrared negatives is available.

Enlargement printing of black and white, true color, and color infrared negatives to various sizes and scales is available. Enlarging capability is from a minimum of 0.3 m square to a maximum of 1.25 m wide by 3.65 m long on a single print in color or black and white on paper or film. Magnification capability is 2X through 100X.

Print Mosaicking, Mounting, and Framing

Mosaicked prints, print mounting, and framing are available. Mounting materials are available in three thickness.

Project Quality

Discuss the project goals with the aerial photo lab personnel. Technical and fiscal assistance is provided in person, by facsimile or telephone. Experience in this discipline can save dollars and time while improving photo image.

Placing An Order

Use the Aerial Photography and Lab Service Request form (DOT 350-148) if it is available. Mail routine requests to the following address:

WSDOT Aerial Photo Lab
PO Box 47384
1655 South Second Avenue
Tumwater, WA 98504-7384

Expedite urgent requests by telephone or facsimile to the Aerial Photo Supervisor.

Field Surveys

Survey telephone (360) 709-5530.

Control surveys are performed as a base for photogrammetric mapping and subsequent use by the field engineers in location and construction projects.

Ground control for aerial photography must be set in accordance with Chapter 1450 of the *Design Manual* and the specifications and procedures in Chapters 4 and 5 of this manual.

Assistance in placing control points, methods of placement, training of personnel, location of known points, accuracies required, and any other help needed to assure a quality product is available from the Geographic Services Branch.

Coordinates and elevations should be submitted either on a disk or as a clearly labeled list. Highly accurate control nets are now available with Global Positioning Survey (GPS) units from the Survey Section.

Premarking

The placing of photographic targets prior to aerial photography will assure that positive identification of the control survey points is made. This will alleviate errors and delays caused by misidentification of “picture points.”

It is strongly recommended that the Photogrammetry Section be consulted for a control scheme designed for a specific project when working in difficult terrain or if the project will have multiple flight lines.

Here are a few suggestions on placing the targets:

1. Place them so they will be away from buildings, trees, and shadows.

2. Overlap flight lines so that ending model control can be used as beginning model control in the next flight. See Figure 7-2.

3. Premark all horizontal and vertical points to be shown on the map, especially points which would be difficult to tie to the control survey.

4. The premark used should contrast with the background it is placed on. This is very important.

5. Premarks may differ in shapes and materials depending on accessibility of area, material available and placement surface. They may be painted with a sprayer or brushed on pavement or gravel, using a template for consistent size and shape. Cloth, paper, or plastic targets may be used on ground, either weighted by rocks or tacked to driven stakes or hubs.

6. Avoid placing targets where they will be disturbed by livestock, wind, farming operations, and traffic. Where necessary, remove targets as soon as photography is completed.

7. Templates for small targets may be made of plywood, fiberboard, or cardboard. Large templates may be built in halves. Refer to Figure 7-3 for fabrication diagram.

8. The placement of extra targets in projects that have obscured areas may help.

The size, shape, spacing, and material used will be dictated by the photography scale, weather conditions, and type of terrain on which the targets will be placed. The accompanying chart in Figure 7-4 illustrates the recommended target dimensions and spacing.

Note: Try not to place cloth targets too early as the target stimulates vegetation growth which distorts the vertical reading.

Standard 1 m cloth targets are available from the Survey Section.

Establishing Control Photogrammetrically

In many instances, it is impossible for field crews engaged in establishing control surveys to reach certain areas due to topography or accessibility. To overcome these situations, equipment is now available to assist the field engineer in obtaining additional control data.

By using a stereoplotter, horizontal and vertical positions of desired points (preferably premarked) can be established to a high degree of accuracy. This method is particularly useful in determining the positions of section corners, property lines, property corners, or any other existing features which are identifiable on the photograph.

Another useful feature of this method is the establishing of horizontal and vertical positions of premarked control points, or other identifiable features, which may surround an area of heavy vegetation. These can then be used by the engineer to accomplish field completion in areas not covered by the photogrammetric mapping.
(See Figure 7-5.)

Programming

The inclusion of mapping, photographic products, or control establishment in the initial programming of a project is essential to form a schedule of activities and services to be performed. This will best serve the interests of the project manager and photogrammetry by establishing priorities and delivery schedules.

Mapping

When photogrammetric maps or cross sections are required, requests for these services should be initiated as early as possible for scheduling purposes. Details can be provided at a later date.

Information to be submitted for establishing a mapping schedule shall include: length of project, mapping widths, type and scale of mapping, priority of sections if any, the preferred seasonal period (date bracketed) for photographing, and the latest date delivery can be accepted.

Special instructions, any deviation from standard work, and mapping limits must be furnished prior to the date of photography.

Photographic Products

Standard ordering procedures described below shall be followed when ordering oblique or vertical aerial photography, mosaics, or enlargements for illustrative purposes.

Field Surveys

Programming for geodetic or control surveys is to be accomplished when mapping projects are scheduled.

Prior to the accomplishment of any control surveys, and preferably at the time the control survey is requested, a sketch (diagram) of the proposed work should be forwarded to the Photogrammetry Branch. This plan should indicate the known existing control stations, proposed primary and supplemental control stations to be established, and tentative mapping limits (if any). GPS and monumentation should also be considered.

Photographic Products and Services

The aerial negative is a source of other products and services available from the Geographic Services Branch.

From an aerial negative, the following products may be obtained:

1. Contact prints. 230 mm by 230 mm paper prints are used for stereoscopic viewing, illustrative use, assembling for mosaics, interpretation, or as records of conditions for special studies.

2. Enlargements from aerial negatives. Enlargements may be furnished on photographic paper in all sizes up to 3.66 m long, unmounted for easy portability, or mounted for display purposes at public hearings and meetings. Art work can be performed on enlargements to indicate proposed design features for "before" and "after" conditions.

3. Enlargements from copy negatives. Copy negatives are made by copying mosaics, maps, or enlargements, and furnishing prints at various scales and sizes. Enlargements of copy negatives can be made up to 5X the original size, and can be furnished on photographic paper or stable base film in continuous tone positive, and printed reverse-reading for easy reproduction.

4. Miscellaneous products. Other useful products available include vacuum-frame printed duplicates of map sheets. These can be furnished on stable base material, sepia, or standard solid type paper. This is very useful when exact duplicates for more accurate scale control are needed for projection and design work.

   Copies of a 100 mm or 250 mm grid on stable base material, prepared accurately, are useful in the plotting of coordinates for projection and design work and can be furnished upon request.

Procedures for Ordering

All requests for photography, mapping, and related items shall be forwarded to the Geographic Services Branch for processing.

Requests should be sent through the Region Photogrammetry liaison person if one is available.

Forms are available for use in requesting any phase of photogrammetric service. The form Request for Geographic Services is available through FileMaker Pro. Figure 7-6 shows the aerial photography form available from Geographic Services. These forms should be submitted with all pertinent data, such as vicinity maps indicating the areas to be covered by photography, mapping, or control surveys; details or premarking operations; and the specific use for which the service or product is required.

The Photography Request form must be used if new photographs are required.

The request for Photogrammetric Services form must be used if mapping, cross sectioning, or DTM is needed.

Note that if both new photography and mapping, and/or cross sectioning are needed, both forms must be used.

The Aerial Photography and Lab Service Request form is used when products, such as prints or enlargements, from existing photographics are desired.

Technical Information

Photograph Scale

To determine the scale of a photograph, divide the average flight height by the focal length of the camera. This will give an approximate average scale of the photograph.

Most mapping cameras use a 152 mm focal length.

Mapping Scale

To determine the mapping scale which may be obtained from photographic work, divide the scale of the photograph by the enlargement ratio of the mapping equipment. A ratio of 5 is usually used (the enlargement ratio of a plotter). However, enlarging is more flexible with today's equipment. For cross sectioning, a ratio of 5 is desirable. A larger ratio may be used for planimetric or topographic maps.

For topographic maps, consider the "c" factor. This is obtained by dividing the flight height by the contour interval. A "c" factor below 1800 is acceptable for location work. A "c" factor below 1000 is preferred for design work.

For example: with a scale of 1:6000, the flight height with a camera having a 152 mm focal length would be 912 m. For a map with a 0.5 m contour interval, the "c" factor would be 912 m/0.5 m = 1824 m. Therefore, a photo scale of 1:3000 would be best for a PS&E map with a 0.5 m contour interval to get the "c" factor below 1000.

Photographic Coverage

To determine the number of photographs required to cover a given distance in stereo:

Photo Scale 1:	**Photos Per Kilometer**
2400	6
3000	5
3600	5
4800	4
6000	3
12 000	2
24 000	2

P:HSM7

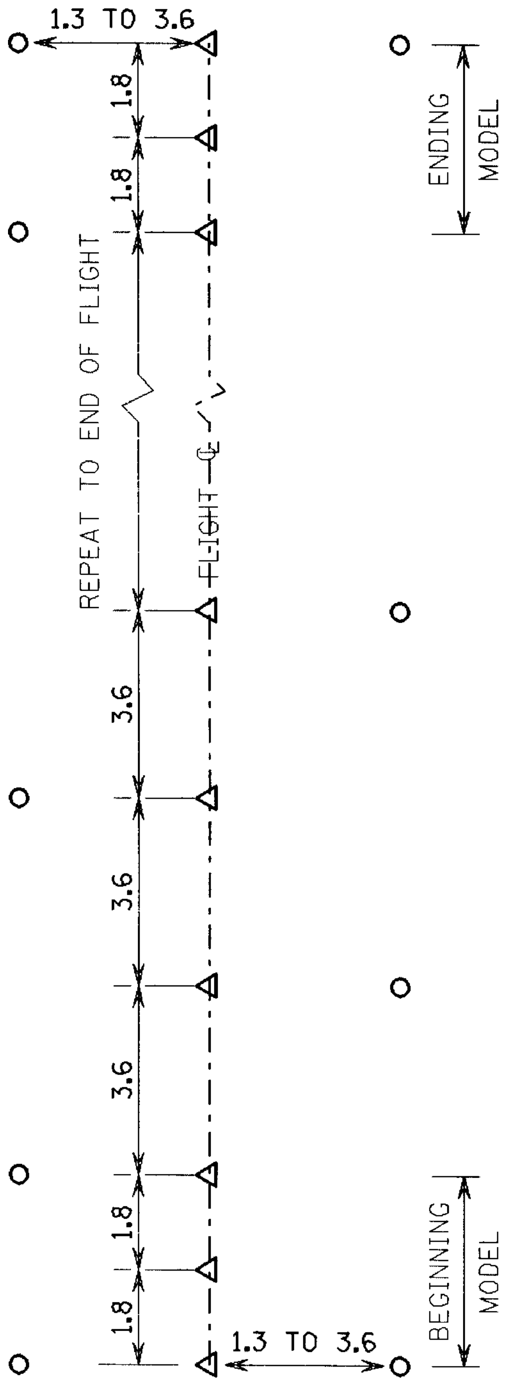
Reference Chart —
Horizontal and Vertical Map Accuracies as Related to Photo Scale

(Approximate values — check with Photogrammetry about specific projects)

Photo Scale	Map Scale	Contour Interval	Target Size	Field Control Accuracy		Map Accuracy		Cross Section Accuracy
				Horz.	**Vert.**	**Horz.**	**Vert.**	**Vert.**
1:24000	1:4800	6 m 3 m	Photo ID 7 m	±0.6 m ±0.6 m	±1 m ±0.6 m	±0.6 m ±3 m	±3 m ±2 m	
1:12000	1:2400	3 m 2 m	3 m 3 m	±300 mm ±150 mm	±0.6 m ±300 mm	±2 m ±2 m	±1.5 m ±0.8 m	
1:6000	1:1200 1:1200 1:600	2 m 0.5 m 0.5 m	2 m Target or Standard Target with legs	±150 mm ±130 mm ±100 mm	±150 mm ±60 mm ±60 mm	±1 m ±0.6 m ±0.6 m	±0.8 m ±0.3 m ±0.3 m	±120 mm
1:3000	1:600 1:480	0.5 m	Standard Target	±80 mm	±60 mm	±0.6 m	±125 mm	±60 mm
	1:600 1:480	0.4 m	Standard Target	±60 mm	±40 mm	±0.5 m	±100 mm	
	1:240	0.5 m 0.4 m	" " " "	±60 mm ±50 mm	±40 mm ±30 mm	±0.4 m ±0.4 m	±125 mm ±80 mm	
1:2400	1:480	0.5 m 0.4 m	" " " "	±50 mm ±40 mm	±40 mm ±30 mm	±0.5 m ±0.4 m	±120 mm ±60 mm	
	1:240 1:120	0.4 m	" "	±30 mm	±25 mm	±0.3 m	±60 mm	

Note:
1. Ninety percent of map will meet or exceed indicated accuracies.
2. Horizontal accuracy requirements are based on relative position.
3. Vegetation cover will decrease all vertical accuracies.
4. The arithmetical mean of 97 percent of the cross section shots shall not exceed the accuracies shown.
5. Standard Targets are 1 m square.

Figure 7-1



NOTES

○ INDICATES WING POINTS.

BEGINNING AND ENDING MODELS REQUIRE A MINIMUM OF FOUR VERTICAL AND THREE HORIZONTAL VERTICAL PREMARKS.

ALL RATIOS X PHOTO SCALE.

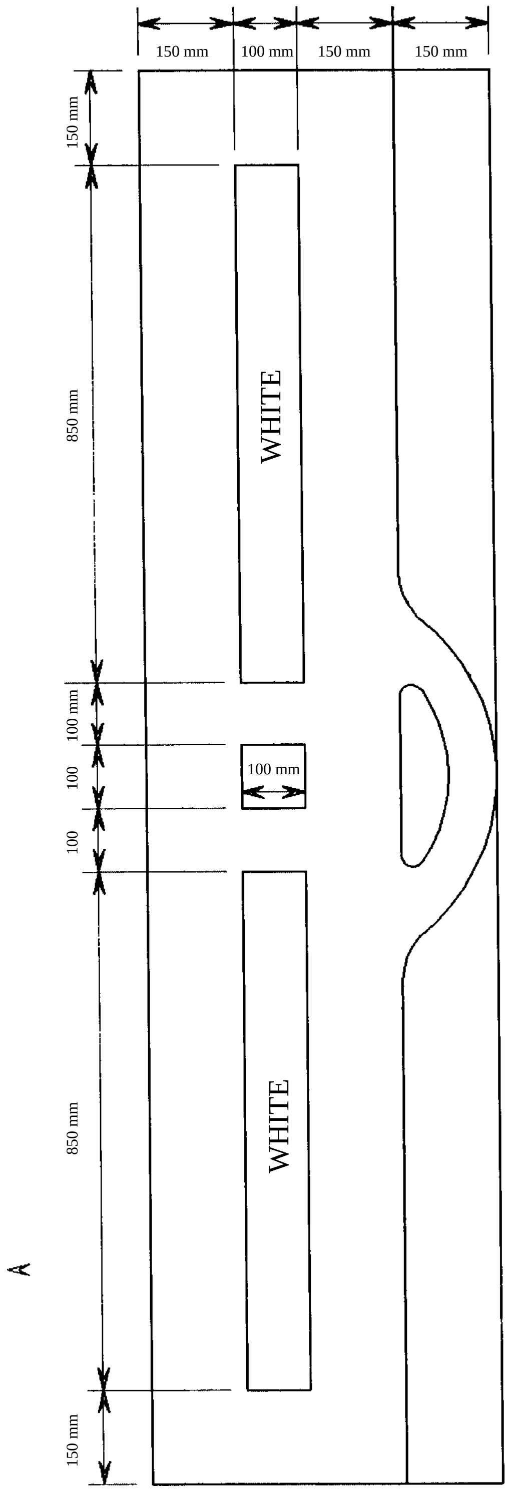
MINIMUM DISTANCE TO WING POINTS TO BE 1.3 X PHOTO SCALE OR BEYOND WORK AREA, WHICHEVER IS GREATER.

THIS IS AN IDEALIZED CONTROL DIAGRAM. IT IS STRONGLY RECOMMENDED THAT THE PHOTOGRAMMETRY UNIT BE CONSULTED FOR A CONTROL SCHEME DESIGNED FOR A SPECIFIC PROJECT WHEN WORKING IN DIFFICULT TERRAIN OR IF THE PROJECT WILL HAVE MORE THAN ONE FLIGHT LINE.

WHEN IN DOUBT WHETHER A TARGET WILL BE VISIBLE, PLACE AN EXTRA TARGET.

Premark Ratios
Figure 7-2

PLYWOOD
10 mm x 550 mm x 2.3 m



See page 7.11 for premark dimensions used on other photo scales.

Instructions for mixing paint —

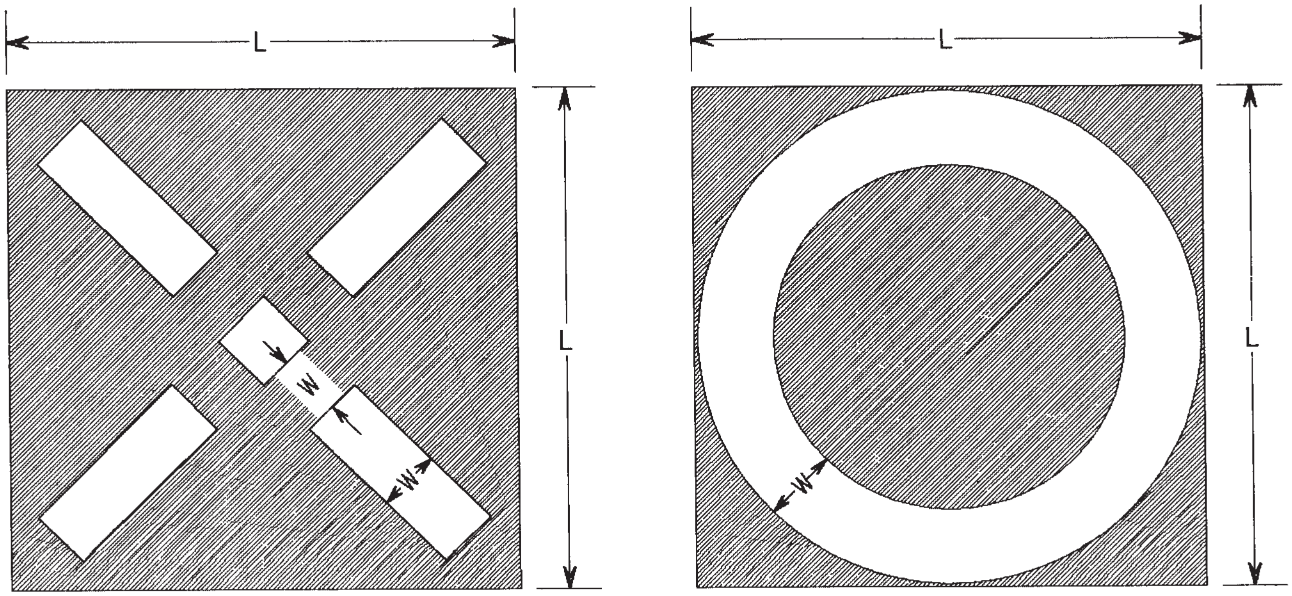
Mix 5 units of white traffic line paint with 1 unit of methyl-ethyl ketone thinner (50-212) to paint center and legs of premark.

For good contrast, paint premark background with black chlorinated rubber base paint.

Premark Paint Template for 1:5000 Photography
Figure 7-3

Vertical Only Points can be Marked for

1:5000
1:10000
1:20000



1 2

Photography Scale	1:2500 or 1:3000		1:5000		1:10000		1:20000	
Target Dimensions	L	W	L	W	L	W	L	W
	1 m	100 mm	2 m	200 mm	4 m	400 mm	6 m	600 mm
Desired Interval Between Targets	*		*		*		*	
Possible Map Scales	1:100 - 1:500		1:500 - 1:1000		1:1000 - 1:2500		1:2500 - 1:5000	

Note: Targets are numbered in order of preference and are used for horizontal and vertical control.

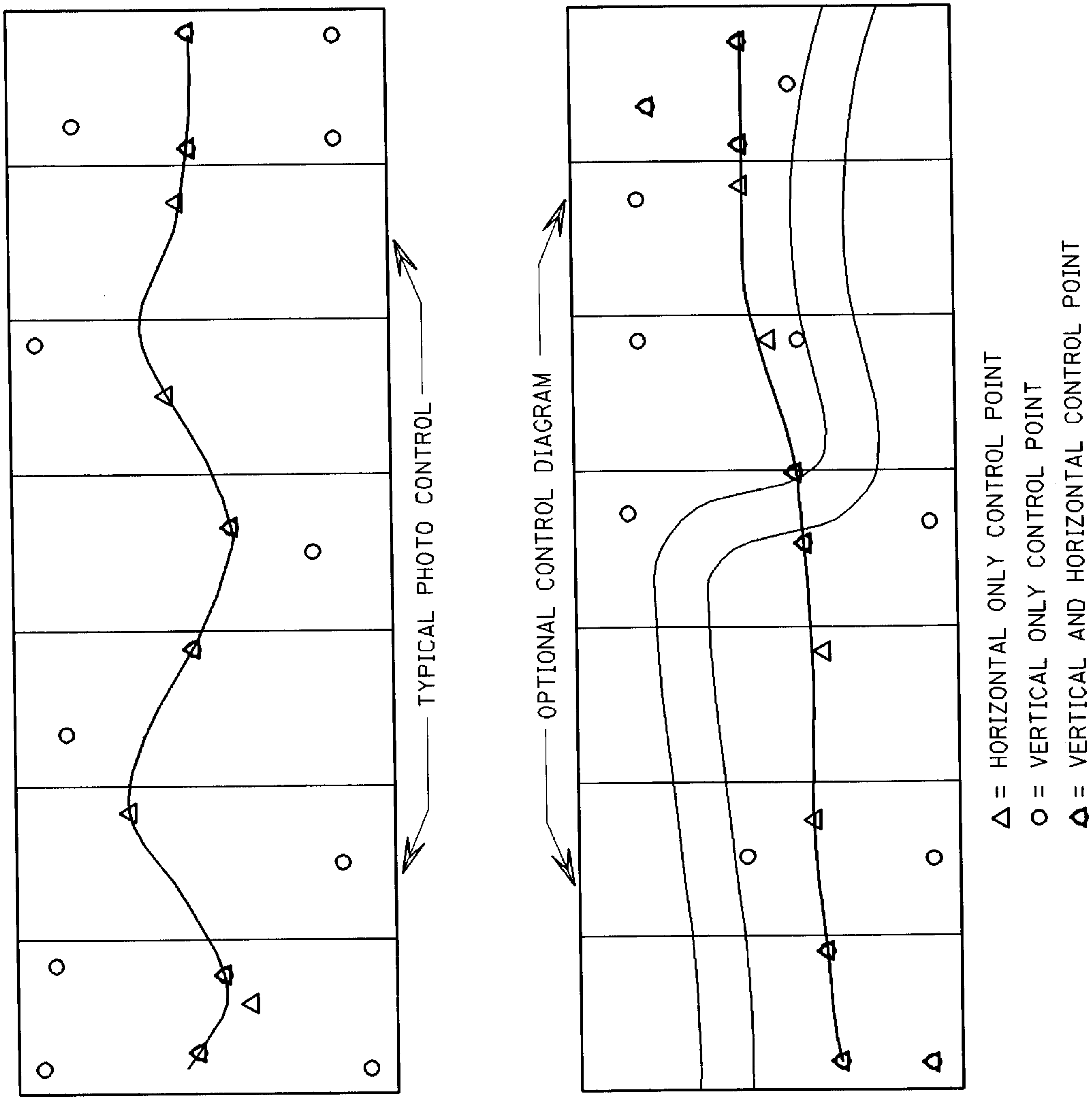
Standard targets are type no. 1 printed on approximately 1 m x 1 m cloth or plastic and are available from the Geographic Services Branch.

Type no. 2 targets, though less desirable, are very effective when marking existing street monuments.

When the target size required is larger thant he standard target, special fabrication or painting must be provided. For photography scales of 1:5000 or 10000, standard targets may be modified by overlaying and extending with legs of the proper dimensions shown in the table above.

* See Premark Ratios layout

Photogrammetric Targets
Figure 7-4



Suggested Photo Control Location
Figure 7-5

Aerial Photography and Photolab Service Request

Customer Contacted
Time | Date

☐ Phone ☐ Mail
☐ FED-X
☐ UPS ☐ Pony Express

Ordered By__________ Date Requested__________ Date Required__________

Signature__________ Org.__________ Phone (   )__________

SR__________ Project Title__________

Job Number	Group	Work Op.	Obj.	Org. Number	Control Sect.	Charges	Invoice Voucher
			TE-75				96–
					Check No.__________	Amount Paid $__________	
					Date__________	Receipt No.__________	

Special Instructions__________

Non-Standard Charge:__________

Aerial Photography

☐ Obliques
☐ Verticals

Photo Scale

1:__________

Lens Type

☐ 6″ Lens
☐ 12″ Lens

☐ Mapping ☐ Non-Mapping

☐ Targets OK__________

Film Type

☐ Color
☐ Color Infrared
☐ B/W
☐ B/W Infrared

Aerial Negatives

____ ☐ Color
____ ☐ B/W

Lab No. 1________ Date__________

Medium / Small Format Photography

☐ One Photographer
☐ Two Photographers

Film Type

☐ Color Slides
☐ Color Negatives
☐ B/W Negatives
☐ Helicopter Hours
☐ Scheduling Fee

Contact Printing

____ ea ☐ Color Prints
____ ea ☐ Color Film Diapositives
☐ Color Balance

____ ea ☐ B/W Film Diapositives
____ ea ☐ B/W Prints

Photo ID No.__________

Lab No. 2________ Date__________

Film Can #__________

Studio Services

Print Mounting

☐ 3/16″ Gator Foam
☐ 3/16″ Foamboard
☐ 1/2″ Gaterfoam
☐ Black ☐ White

Print Laminating

☐ Clear
☐ Matte
☐ Matte UV

Other

☐ Black tape edges
☐ Frames
☐ Mosaic
☐ Splice
☐ Labor

Lab No. 5________ Date__________

Enlargement Printing

____ ea ☐ Color Paper
☐ Color Balance

____ ea ☐ B/W Paper
____ ea ☐ B/W Film
☐ Matte
☐ Clear
☐ Opaque

Enlargement Factor__________X

Scale 1:__________

Photo ID No.__________

Lab No. 3________ Date__________

Film Can #__________

Customer Services

☐ Notarized Letter for Court Exhibits
☐ Scale Ratio
☐ Negative Retrieval
☐ Lettering

Twin Check #__________

Ship To__________

Geographic Services
DOT Form 350-148
Revised 3/96

White — Photo Lab Canary — Billing Pink — Return to Customer

Figure 7-6

8

Public Lands Survey System

History and Background

The Public Lands Survey System (PLSS) was originally set up to sell the lands acquired by the U.S. Government in the Northwest Territory (Ohio, Indiana, Illinois, Michigan, and Wisconsin). It was so successful that it was used later to sell land in the Louisiana Purchase, Oregon Territory, Mexican Cession, and Alaska.

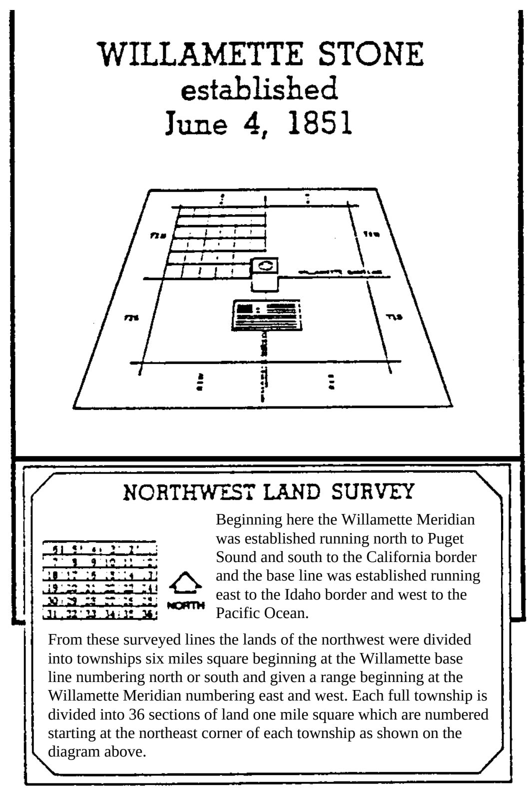
The system designates land location east or west of a principal meridian and north or south of a base line. In the state of Washington, the principal meridian is known as the Willamette Meridian (abbreviated as W.M. in legal descriptions). The principal meridian runs north and south from a stone monument established in 1851 located in Portland, Oregon. This stone monument is also on the east-west base line (the Willamette Base Line). See Figure 8-1.

Distances north or south of the base line are described by east-west townships lines. The township lines are intended to be at approximately six-mile intervals. A point eight miles north of the base line would then be in the second township north of the base line and would be written as being in Township 2 North, abbreviated as T2N.

Distances east or west of the principal meridian are described by north-south range lines at approximately six-mile intervals. A point eight miles east of the principal meridian would be in the second range east and would be written as Range 2 East, abbreviated as R2E.

The combined description of the point would then be in Township 2 North, Range 2 East or T2N, R2E.

The Willamette Base Line runs east and west on a true parallel of latitude. To keep the township lines parallel to the base line, *standard parallels* were established at 24-mile intervals north and south of the base line. Similarly, guide meridians were established at 24-mile intervals east and west of the principal meridian. Since meridians converge towards the North Pole, the further the range lines are from the principal meridian, the more they



Willamette Stone
Figure 8-1

diverge from true north. The guide meridians establish a new starting meridian for the next set of townships. The beginning point of the guide meridian is on the standard parallel or base line and the ending point is at its intersection with the next standard parallel.

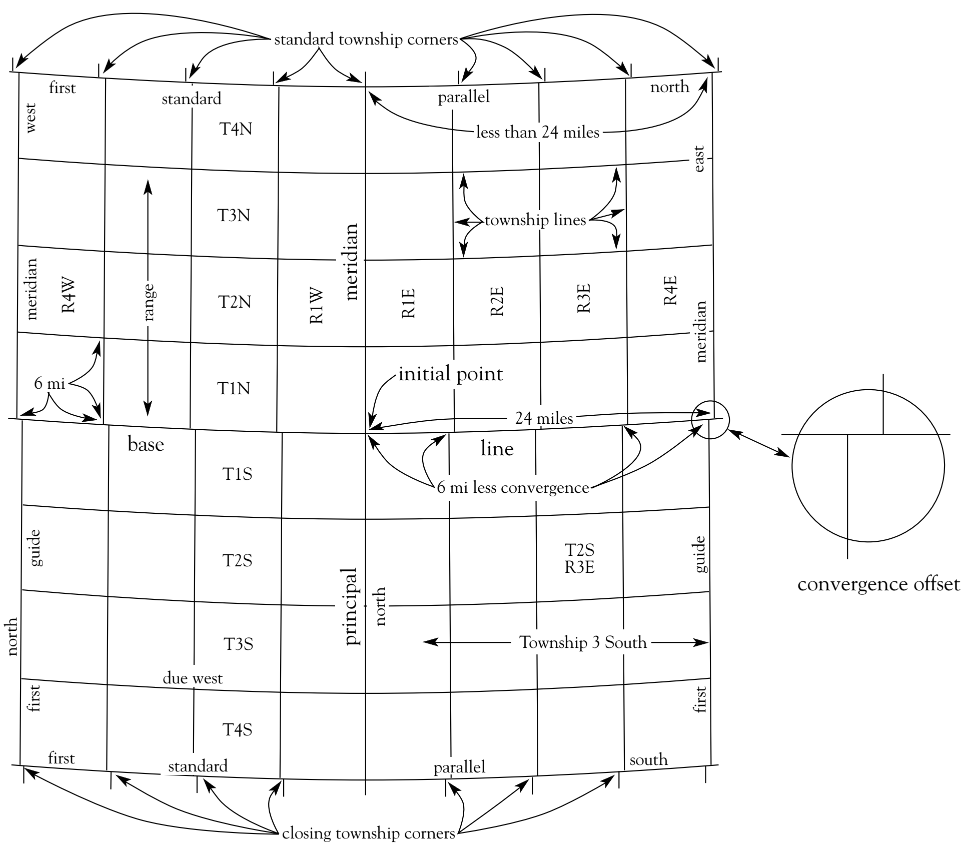
Thus, on the base line and on a standard parallel, there will be a convergence offset between the guide meridian to the south and the one to the north.

The final result is a series of quadrangles bounded by meridians and parallels each including 16 townships. See Figure 8-2.

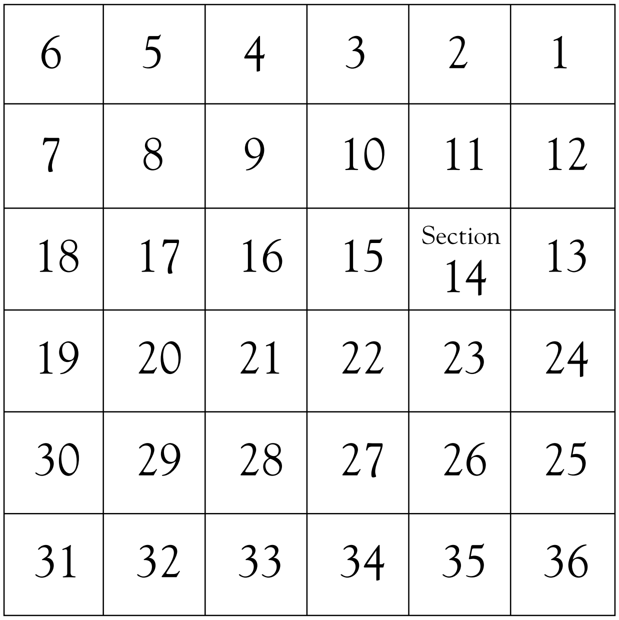
Township Subdivision

Each township, approximately 6 miles on a side, was further divided into 36 sections and monuments were set at the section corners. Each section was intended to be 640 acres or one square mile in area. Because of the relatively crude instruments used at the time and the varying capability of the surveyors who laid out the sections, it is unlikely that any section is actually that size.

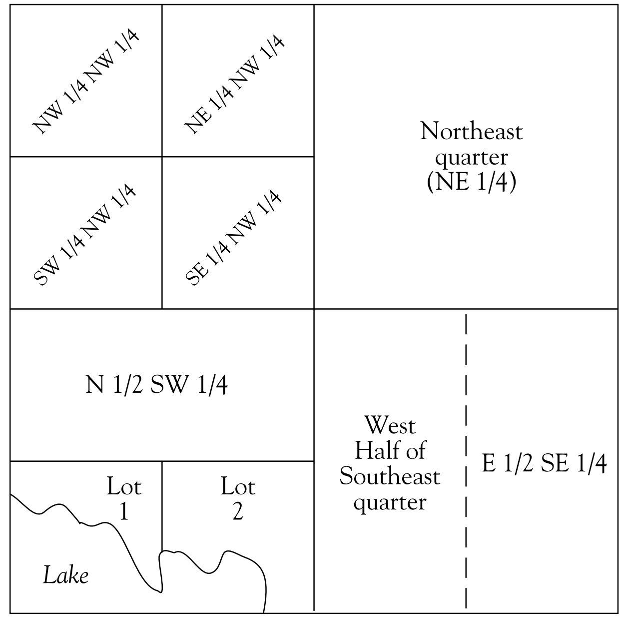
The numbering of the sections starts at the northeast corner of the township with Section 1 (Figure 8-3) and progresses westerly to Section 6 at the northwest corner of the township. Section 7 is south of Section 6 and the numbering then increases to the east with Section 12 directly south of Section 1. The numbering continues, alternating between increasing to the east and increasing to the west, then to Section 36 in the southeast corner of the township.



Principal Meridian, Base Line,
Standard Parallels and Guide Meridians
Figure 8-2



Township Grid
Figure 8-3



Section Grid
Figure 8-4

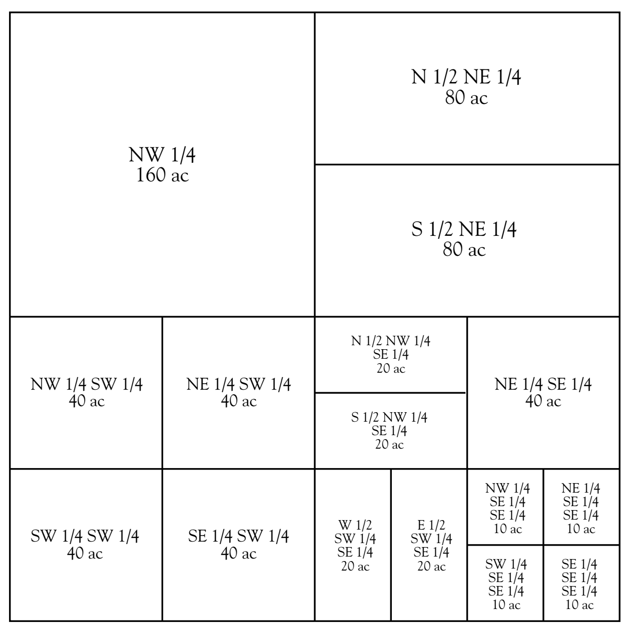
The dimensions of the sections were measured in chains and links. A chain of 100 links is equal to 66 feet and a link is 7.92 inches. A section was intended to be 80 chains (one mile) on each side.

Section Subdivision

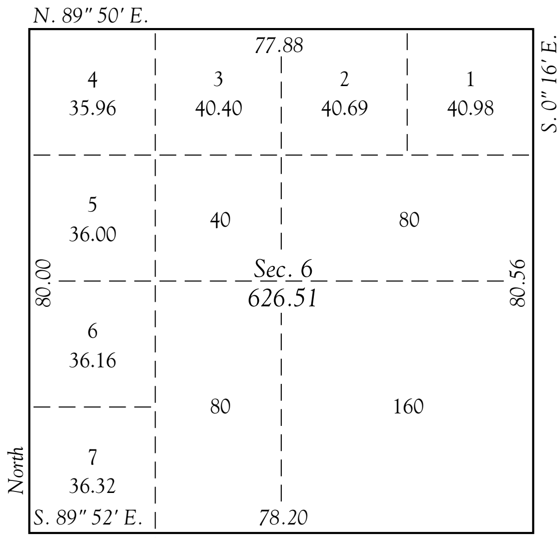
Along with the section corner monuments, quarter corner monuments were set at the halfway point on each side of a section. Thus, the 80 chain sides were theoretically divided into 40 chain (1/2 mile) segments. The original surveyors did not set a monument at the theoretical center of section (where the lines connecting quarter corners would intersect). Each quarter of a section (1/4 square mile) was theoretically 160 acres and could be divided again into 80-acre halves or 40-acre sixteenths. See Figures 8-4 through 8-7.

Certain areas of land were not available for sale and the boundaries of the Public Lands Survey System (PLSS) stopped at the edge of these areas. Some examples are:

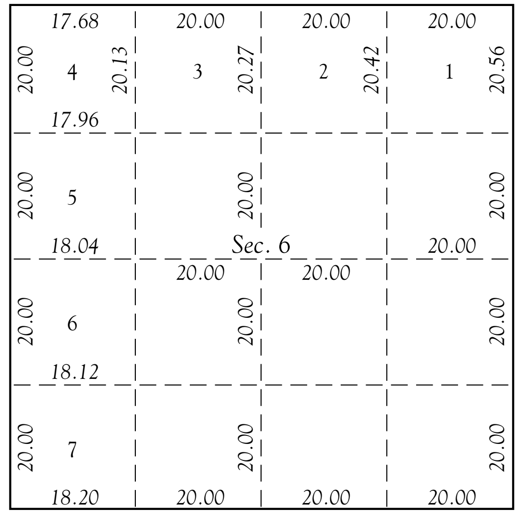
- Indian reservations (IR)
- Donation land claims (DLC)
- Military reservations (MR)



Nomenclature for Portions of a Section
Figure 8-5



Showing areas in acres



Showing calculated distances in chains

Example of Subdivision of Section 6
Figure 8-6

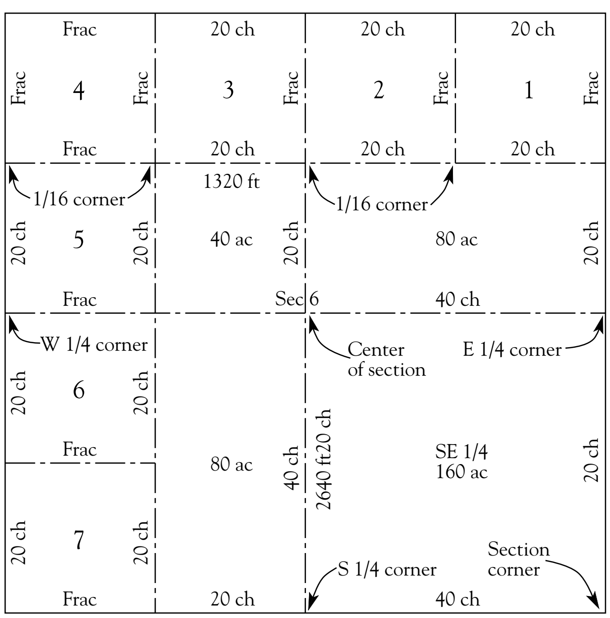
- Some bodies of water
- Mining claims

To establish where the PLSS lines intersected the boundary line, auxiliary corners were set, known as witness corners (WC).

Because of the errors in the original surveying, when the surveyors started to break down a township into sections, they often found that the dimensions were more or less than six miles. The usual practice in this state was to try to keep the error of closure on the north and west sides of the township. This could mean that the sections on the north and west sides would not be 640 acres. To try to keep the quarter sections close to 160 acres, any odd distances in the section were also kept on the north and west sides.

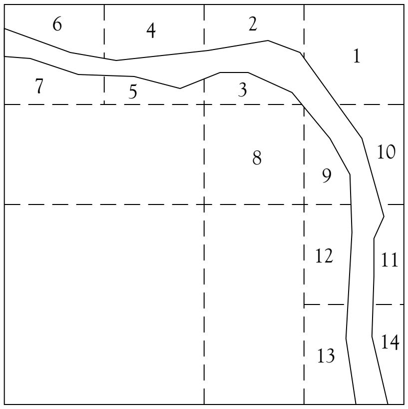
The section was broken down into regular quarter sections as much as possible and the irregular pieces were called Government Lots (GL). This same system was also used where the section intersected one of the reserved areas listed above (IR, MR, DLC, etc.). See Figure 8-8.

In summary, a section is approximately one square mile unless it lies on the north or west side of a township or abuts one of the areas not available for sale. In these irregular sections, there will be a mixture of regular quarter sections and Government Lots.

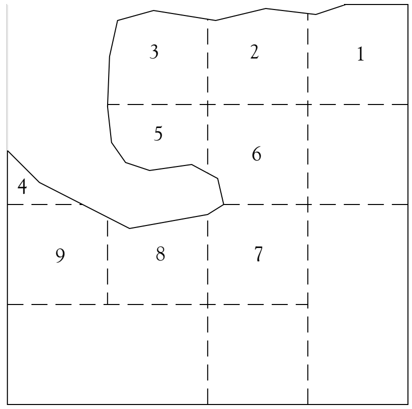


Section 6 has Fractional Measurements of more or less than 20 Chains on Lines Marked "Frac."

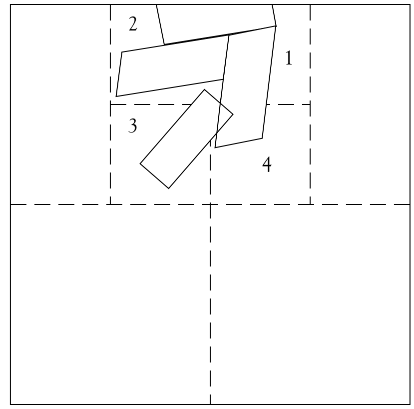
Fractional Measurements
Figure 8-7



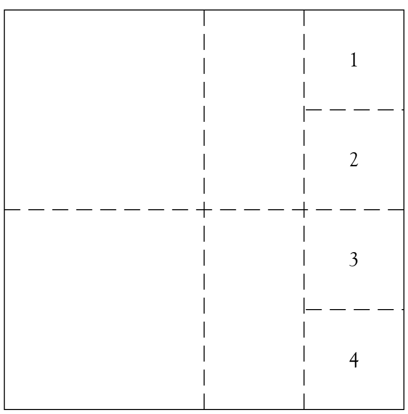
Meanderable River



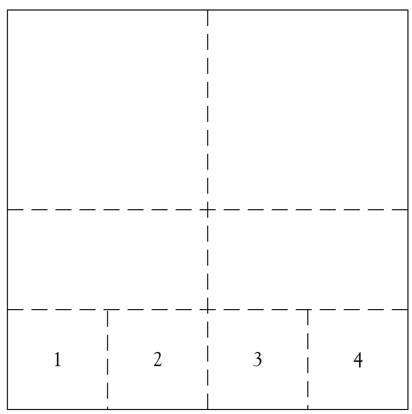
Meanderable Lake



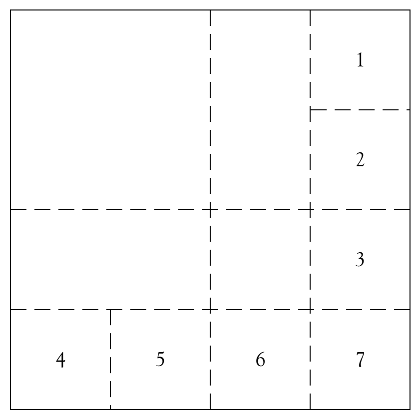
Mineral Claims



East boundary
defective alignment



South boundary
defective



East & South boundaries
defective

Example of Government Lots in Irregular Sections
Figure 8-8

The surveyors used whatever was available as a monument. Where wood was available, they used a post, usually 4″ × 4″, marked on the side with the section designation. Where wood was not available, they used stone monuments, or dug a pit to create a mound, or buried a heap of charcoal. See Figure 8-9.

Since the original surveys, many of the original monuments have been perpetuated by government agencies or private surveyors with more permanent markers.

In addition to monumenting the corners, the original surveyors also marked trees along the boundaries when available. These are known as line trees and were blazed or hacked with an axe. See Figure 8-10.

Similarly, if trees were near a corner, the surveyor recorded the distance and bearing of the tree, removed a piece of bark, and marked on the living wood tissue. Frequently, the bearing tree (BT) has been cut down but the stump may still be there. Since the section corner was also the meeting place of four property lines, it was frequently used as a fence corner. By using the recorded bearings and distances from bearing trees or other reference points, it is possible to verify if an old fence corner was placed at the location of the original section corner. See Figures 8-11 and 12.

In summary, almost anything might have been used to replace an old corner. The problem is then to confirm that what is found at the supposed site of the corner is at the exact location of the old corner and not just in the vicinity. If there is a record in a survey book that some surveyor replaced the original corner (for example, “a rotten post scribed with proper markings”) with an iron pipe, the problem is then to determine if the iron pipe that is found is the same one that was set. If it is possible to measure from other reference points and check the distances, then it is reasonable to accept the pipe as a replacement of the original corner. However, if there is no record of the replacement, the fact that there is an iron pipe does not automatically make it the corner. Any monument is subject to dispute and it is necessary to prove the position of anything other than the original corner. Since the original surveys were relatively crude, the recorded distances in chains between corners may not agree with the actual distances. However, once the surveys have been accepted and the land sold, the position of corners as set originally in the field will control over described position.

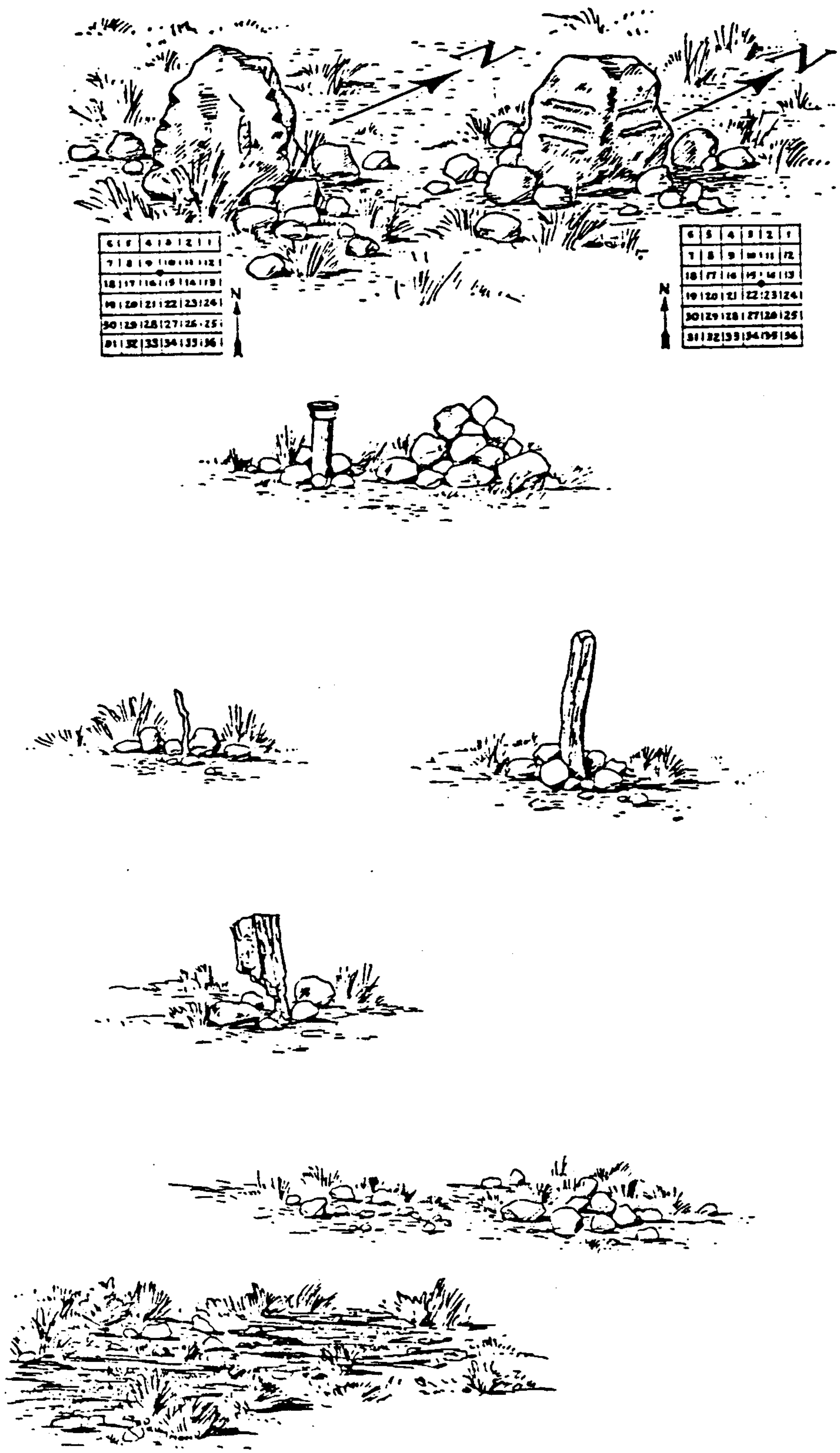
If the corner (or its replacement) is missing, the corner may be either “lost” or “obliterated.” An obliterated corner is one that has been destroyed but whose position can be reestablished from other recorded reference points nearby.

A lost corner is one where all evidence of its location is gone. The location can be reestablished by rerunning the original survey. A corner is not considered lost if it has been established using the State Plane Coordinate System.

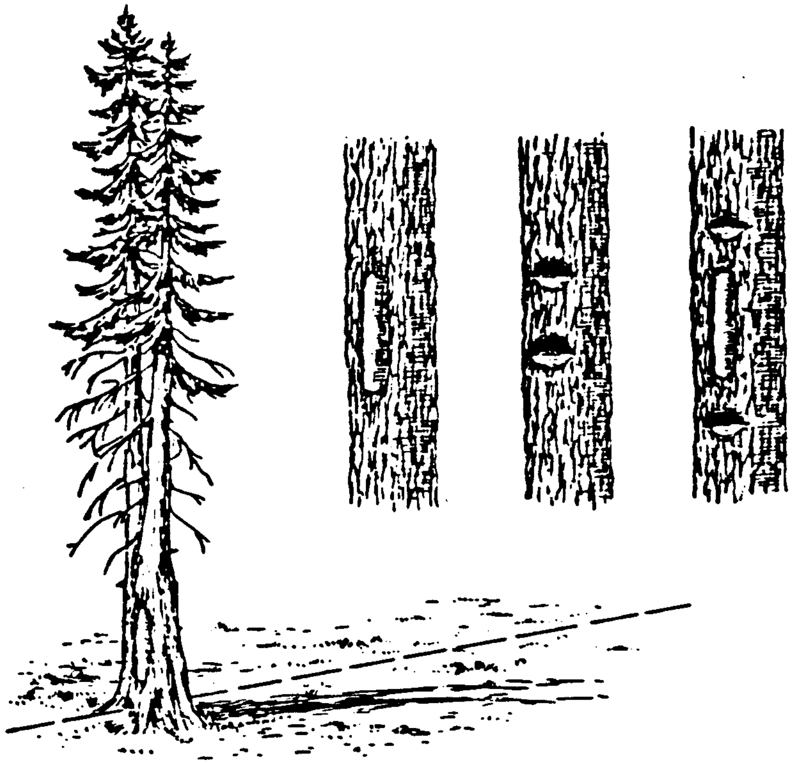
Restoring Lost or Obliterated Corners

Restoration of a lost or obliterated corner must be done according to specific rules. See Chapter 9.

P:HSM8

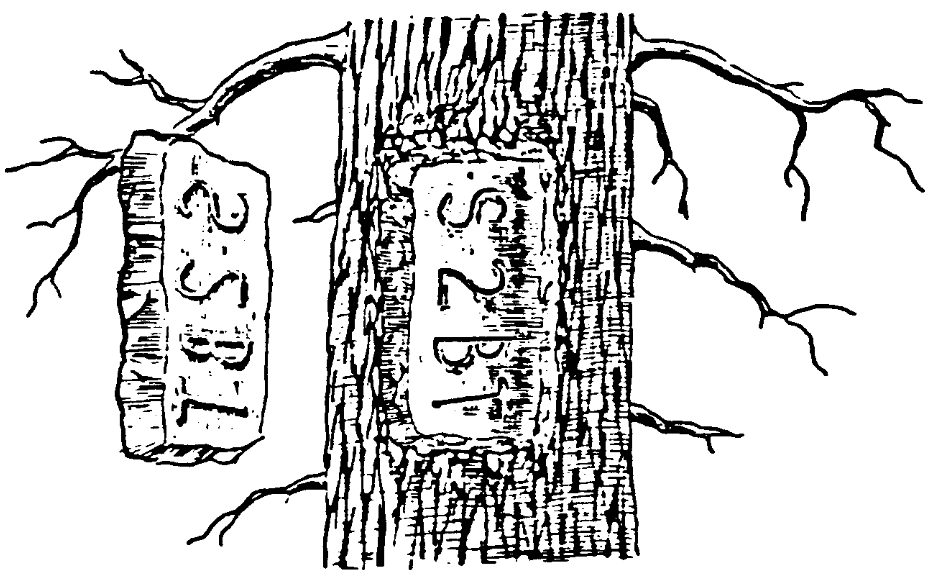


Corner Monuments of the Public Land Surveys
Figure 8-9



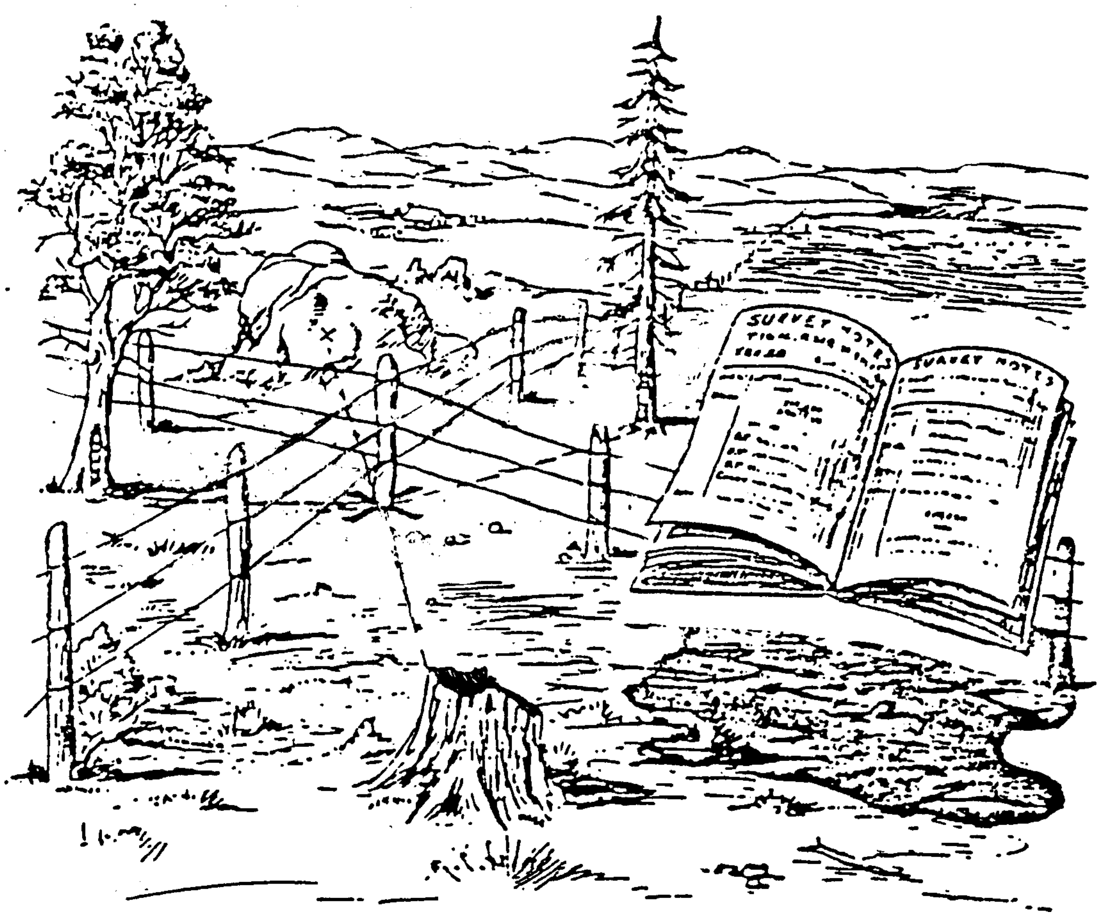
Marks Found on a Line Tree may be a Blaze, a Hack, or Both

Line Tree
Figure 8-10



The original marks are preserved and appear in reverse and relief on the overgrowth

Old Bearing Tree with Overgrowth Removed
Figure 8-11



The position for a corner of the public land surveys may be recovered
by reference to the recorded bearing trees or bearing objects

Bearing Objects
Figure 8-12

9

Monuments

Monuments are defined in Washington Administrative Code (WAC).

WAC 332-120-020 Definitions.

Survey monument: The physical structure, along with any references or accessories thereto, used to mark the location of a land boundary survey corner, geodetic control point, or local control point.

Examples of monuments are brass caps, brass plugs, marked stones, iron pipes, lead plugs with tacks, wooden hubs with tacks, railroad spikes with punch marks, finishing nails or brass screws in fence posts, PK nails, concrete posts, reinforcing bars with plastic or aluminum caps and holes drilled in rocks.

Monuments set by government agencies are usually identified in some way, either by markings on the monuments or by being in a case identified as belonging to a government agency.

Monuments can be divided into five groups.

1. Horizontal and vertical control – usually set by government agencies.
2. Public Lands Survey System (township lines, range lines, section and quarter section corners, witness corners, meander corners) – originally set by government surveyors. Many have been perpetuated, either by other government surveyors or by private surveyors. If the replacement is not properly marked and recorded, there may be a question as to whether what is found is a genuine replacement or just something in the vicinity.
3. Highway, road, and street alignment – usually set by government agencies.
4. Right of way base line (highway, power line, pipeline, etc.) – set by government or private surveyors.
5. Property corners – usually set by private surveyors.

Monumentation and Description

All primary horizontal and vertical control stations shall be monumented and described.

A monument can be both a horizontal and a vertical control point.

Monuments set by other agencies can be used if they are in a good location. For example, an existing U. S. Geological Survey bench mark could be used for a horizontal control monument.

Specifications for horizontal and vertical control monuments shall meet the standards of the Federal Geodetic Control Subcommittee (FGCS) of the Federal Geographic Data Committee (FGDC). (See Chapters 4 and 5.)

When writing the station description, include information on how to reach the general location of the monument from some prominent feature such as a highway intersection. Include the location by section-township-range and by highway kilometer post (or milepost). Relate the monument to nearby permanent objects by distance and direction and describe the specific details of the monument including any stamping or lettering.

The actual monument is a metal cap, stamped with an agency name and individual identification, firmly set in the ground. (See the Standard Plans.)

Set monuments in firm ground and away from the traveled way so that they can be occupied without danger from traffic. Consider the possibility of frost heave or ground settling.

Place a witness post near the monument to help in finding it again at a later date and to protect it from accidental destruction.

Horizontal Control Monuments

Horizontal control monuments must be measured in the NAD83/91 metric coordinate system of the Washington State Coordinate System.

Primary horizontal control monuments must be tied to the High Precision Network (HPN). Primary horizontal control monuments are normally measured by using the Global Positioning System (GPS). If GPS is not used or is not practicable, then conventional means must be used to establish primary control that is tied to the HPN using procedures and equipment meeting second order class II FGCS specifications (Chapter 4).

Secondary control, designed to supplement the primary control, is used for photogrammetry and right of way surveys, to provide the basis for topography, or the layout of alignments and structures. Secondary horizontal control monuments must be tied to the primary horizontal control monuments by a traverse using procedures and equipment meeting third order FGCS specifications (Chapter 4).

Wherever possible, when setting horizontal control points, place another horizontal control point to act as an azimuth point. Place it so that development in the area will not block the line of sight and in an area that is unlikely to be disturbed by impending construction.

Vertical Control Monuments

Vertical control monuments must be measured in the metric NAVD88 system.

Primary vertical control monuments must be tied to National Geodetic Survey (NGS) bench marks by using equipment and procedures meeting second order FGCS specifications (Chapter 5).

Secondary vertical control, designed to supplement the primary control, is established throughout the project to provide the basis for topography, the layout of grades and structures, or photogrammetry. Secondary vertical control monuments must be tied to the primary vertical control monuments by using equipment and procedures meeting third order FGCS specifications (Chapter 5).

Documentation

The best evidence of a monument's original position is a continuous chain of history by acceptable records, usually written, back to the time of the original monumentation. As part of this, WSDOT surveyors must contribute to the body of public records by documenting monument work appropriately and by striving to preserve and perpetuate existing monuments. See Figure 9-1 and *Design Manual* Chapter 1450.

Deeds have a chain of title back to their inception, and the validity and correctness of a deed is based upon these records. Similarly, monuments should have a continuous chain of history. The original surveyor sets a stone mound for the section corner. Surveyor number two finds the stone mound and sets a 2-inch iron pipe. Surveyor number three finds the 2-inch pipe and sets reference points 30 feet on each side of a new proposed road. Surveyor number four finds the reference monuments and resets the true section corner in the center line of the new road. Surveyor number five finds the new monument in the center line and wants to prove its identity and the correctness of its position. How can he without a continuous record of what each surveyor did? This is the reason that Washington has a law making it mandatory to file a record of survey under certain circumstances.

RCW 58.09.120 requires that any monument set by a land surveyor be permanently marked or tagged with the certificate number of the land surveyor setting it.

SET NEW

WSDOT Control Monument

Before: No permit required.

After: File a Record of Monuments and Accessories with the county engineer, Geographic Services Branch, and DNR.

Alignment Monument

Before: No permits required.

After: Submit Record of Monuments and Accessories to the county engineer, Geographic Services Branch, and DNR.

Property Corner Monument

Before: Engage a licensed professional land surveyor.

After: Licensed professional land surveyor files Record of Survey with county auditor, Geographic Services Branch, and DNR.

DISTURB EXISTING

Control Monument

Before: Obtain DNR permit.

After: File Record of Monuments and Accessories with the county engineer, Geographic Services Branch, and DNR.

Alignment Monument

Before: Obtain DNR permit.

After: File Record of Monuments and Accessories with the county engineer, Geographic Services Branch, and DNR.

Section Corner, BLM, or GLO Monument

Before: Obtain DNR permit.

After: File Land Corner Record with the county auditor, Geographic Services Branch, and DNR.

All Other Monuments

Before:

- Obtain DNR permit.
- Contact governmental agency.

After: File Record of Monuments and Accessories with the county engineer, Geographic Services Branch, and DNR.

Note: See the *Design Manual* for further details and sample forms.

Monument Documentation Summary
Figure 9–1

Monuments set by a public officer shall be marked by an official designation. Prior to this act in 1973, many surveyors did not mark their monuments so it is sometimes difficult to determine whether an object is a monument or not.

History or chain of record for monument position is valuable evidence, but all too often there is an interruption in the history, and a continuous chain of records cannot be proven.

Many disputes have been caused because a monument was not properly identified when it was set or because its replacement was not properly identified and marked. Accidental or deliberate destruction of a monument can lead to long and expensive disputes about where it was originally located. Since many of the old monuments were set when surveying equipment was relatively crude, any attempt to replace a missing monument by relying on old measurements can never put it back in exactly the same location as the original point.

It is therefore essential that, if an existing monument must be disturbed for any reason, its present location must be accurately referenced with modern equipment and methods so that it can be replaced in its old position, or its location accurately determined if it cannot be replaced for some reason.

Since many of the old highways were established along section lines, the original section and quarter section corners were buried under a road or destroyed. Many of these corners were replaced by some type of monument when the road was built. RCW 36.86.050 and 47.36.010 respectively require county engineers and the highway commission to place permanent markers at all public lands survey system monuments where they fall inside the right of way of a county road or state highway. Field books, right of way maps and contract plans may show what was found at the original monument site and what was used to replace it. In general, if county and state maps show a section or quarter section monument in a road or highway, it is reasonable to accept the existing monument as being a replacement of the original point. However, this should be verified by tracing the monument back through the records as far as possible. Due to resurfacing and repaving, the replacement monument may be buried under several layers of paving. Inspection of old plans may indicate where to dig in search of the old point.

State Responsibility – RCW 47.36.010

Restoration of United States survey markers. *The department shall fix permanent monuments at the original positions of all United States government monuments at township corners, section corners, quarter section corners, meander corners and witness markers, as originally established by the United States government survey whenever any such original monuments or markers fall within the right of way of any state highway, and aid in the reestablishment of any such corners, monuments, or markers destroyed or obliterated by the construction of any state highway by permitting inspection of the records in the department's office.*

Removing and Resetting Monuments

Due to deterioration of the original monument or construction at the monument location, it may be necessary to remove the existing monument and replace it later. And, if a primary control monument is to be disturbed, it is impractical to set temporary reference points for resetting it as it is difficult to measure to the required accuracy. The recommended procedure is to set a new monument and then measure the new coordinates and elevation.

Washington Administrative Code (WAC) 332-120 prescribes the procedure for obtaining a permit from the DNR to remove a monument. The DNR Bureau of Surveys and Maps is located at 1102 South Quince Street, Olympia, WA 98504. A copy of the WAC may be obtained through the Transportation Library.

The DNR permit uses the terminology "remove or destroy" as required by law and legally defined as follows:

WAC 332-120-020 Definitions.

Removal or destruction: The physical disturbance or covering of a monument such that the survey point is no longer visible or readily accessible.

The purposes of the regulations are:

- To set standards for the work,
- To establish a chain of evidence on the replacement of an original monument, and
- To ensure that the work is done by qualified personnel who know the correct procedure.

Before removing a monument, the owning agency should be contacted to coordinate the work. Some agencies may prefer to do the removal and replacement themselves. This particularly applies to horizontal and vertical control monuments belonging to U.S. Government agencies.

The documentation consists of:

- Application for a permit, explaining the necessity for removal.
- Permit approval.
- Report on the removal and what was used for replacement.
- Land Corner Record; used when a section corner, quarter corner, street monument, or other point, which is used for locating property lines, is established or replaced and is not reported on some other recorded map or survey. This establishes a chain of evidence which can be used later to prove the genuineness of the monument at that point.

See the *Design Manual* for examples of paperwork for permits to temporarily remove or destroy monuments.

When monuments are set for a new project or to replace old monuments, they should:

1. Be of a permanent nature,
2. Be clearly identified as to who set them,
3. Be referenced to other points so they can be easily reset if disturbed or destroyed,
4. Be recorded with the Department of Natural Resources, the Geographic Services Branch, and with the county auditor or county engineer, depending on legal requirements, so there is a public record of the location and references, and
5. Be located on the Washington coordinate system so they will extend the survey network in this state.

If a monument is offset from the point that it is intended to represent, the record must show the offset and direction between the point and the monument. For example, the base line for a highway may lie along the center of the paving, while the monumentation for the base line may be located in the shoulder.

Public knowledge of the location of the monuments and their relation to the point that they represent will eliminate many problems that have occurred in the past where monuments were destroyed and there was no public record that showed references to remaining points.

P:HSM9

10
Circular Curves

A circular curve is a segment of a circle — an arc. The sharpness of the curve is determined by the radius of the circle (R) and can be described in terms of "degree of curvature" (D). Prior to the 1960's most highway curves in Washington were described by the degree of curvature. Since then, describing a curve in terms of its radius has become the general practice. Degree of curvature is not used when working in metric units.

Nomenclature For Circular Curves

POT	Point on tangent outside the effect of any curve
POC	Point on a circular curve
POST	Point on a semi-tangent (within the limits of a curve)
PI	Point of intersection of a back tangent and forward tangent
PC	Point of curvature - Point of change from back tangent to circular curve
PT	Point of tangency - Point of change from circular curve to forward tangent
PCC	Point of compound curvature - Point common to two curves in the same direction with different radii
PRC	Point of reverse curve - Point common to two curves in opposite directions and with the same or different radii
L	Total length of any circular curve measured along its arc
L_c	Length between any two points on a circular curve
R	Radius of a circular curve
Δ	Total intersection (or central) angle between back and forward tangents
DC	Deflection angle for full circular curve measured from tangent at PC or PT
dc	Deflection angle required from tangent to a circular curve to any other point on a circular curve
C	Total chord length, or long chord, for a circular curve
C'	Chord length between any two points on a circular curve
T	Distance along semi-tangent from the point of intersection of the back and forward tangents to the origin of curvature (From the PI to the PC or PT)

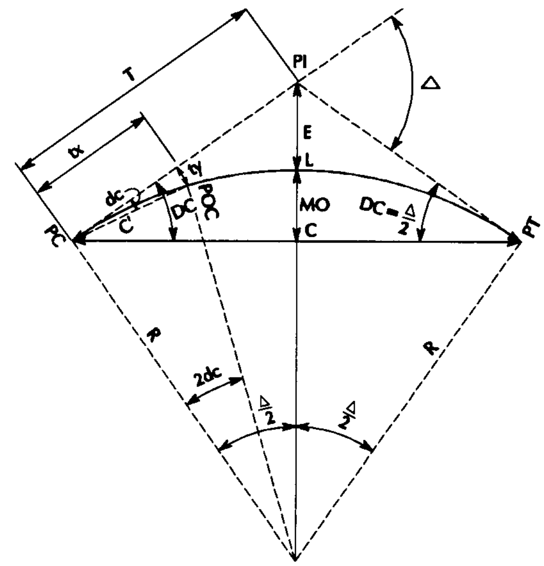
tx Distance along semi-tangent from the PC (or PT) to the perpendicular offset to any point on a circular curve. (Abscissa of any point on a circular curve referred to the beginning of curvature as origin and semi-tangent as axis)

ty The perpendicular offset, or ordinate, from the semi-tangent to a point on a circular curve

E External distance (radial distance) from PI to midpoint on a simple circular curve

Circular Curve Equations

Equations	Units
$\mathbf{R} = \frac{180°}{\pi} \cdot \frac{L}{\Delta}$	m or ft.
$\Delta = \frac{180°}{\pi} \cdot \frac{L}{R}$	degree
$\mathbf{L} = \frac{\pi}{180} \cdot R\Delta$	m or ft.
$\mathbf{T} = R \tan \frac{\Delta}{2}$	m or ft.
$\mathbf{E} = \frac{R}{\cos \frac{\Delta}{2}} - R$	m or ft.
$\mathbf{C} = 2R \sin \frac{\Delta}{2}, \text{or} = 2R \sin DC$	m or ft.
$\mathbf{MO} = R\left(1 - \cos \frac{\Delta}{2}\right)$	m or ft.
$\mathbf{DC} = \frac{\Delta}{2}$	degree
$\mathbf{dc} = \frac{L_c}{L}\left(\frac{\Delta}{2}\right)$	degree
$C' = 2R \sin(dc)$	m or ft.
$C = 2R \sin(DC)$	m or ft.
$tx = R \sin(2dc)$	m or ft.
$ty = R[1 - \cos(2dc)]$	m or ft.



Constant for π = 3.14159265

Simple Circular Curve
Figure 10–1

Curve Stationing

After the length of the curve (L) and the semi-tangent length (T) have been computed, the curve can be stationed. When the station of the PI is known, the PC station is computed by subtracting the semi-tangent distance from the PI station. (Do not add the semi-tangent length to the PI station to obtain the PT station.) Once the PC station is determined, then the PT may be obtained by adding L to the PC station.

All stationing for control is stated to one thousandth of a meter. Points should be set for full stations and at 20 m intervals. Full stations are at 1000 m intervals (1+000.000).

Example Calculations for Curve Stationing

Given: (See Figure 10-1)

PI = 1 + 278.230

R = 500 m

Δ = 86° 28'

Find the PC and PT stations

Calculate T

$$T = R \tan(\Delta/2)$$
$$= 500 \tan 43° 14'$$
$$= 470.079 \text{ m}$$

Calculate L (Δ must be converted to decimal degrees)

$$D = 86° 28'$$
$$= 86° + (28°/60)$$
$$L = \frac{\pi}{180} R\Delta$$
$$= 754.564 \text{ m}$$

Calculate the PC station

PI - T = 1278.230 - 470.089

= 808.151 m

PC station is 8 + 08.151

Calculate the PT station

PC + L = 808.151 + 754.564 m

= 1562.715

PT station is 1 + 562.715

Deflections

To lay out a curve it is necessary to compute deflection angles (dc) to each station required along the curve. The deflection angle is measured from the tangent at the PC or the PT to any other desired point on the curve. The total deflection (DC) between the tangent (T) and long chord (C) is Δ/2.

In the equation

$$dc = \frac{L_c}{L}\left(\frac{\Delta}{2}\right)$$

dc and Δ are in degrees

Example Calculations for Curve Data

Given:

PI = 10 + 000.000

R = 1100 m

Δ = 16° 30'

Find the deflection angles through the curve

Calculate T

$$T = R \tan(\Delta/2)$$
$$= 1100 \tan 8° 15'$$
$$= 159.492 \text{ m}$$

Calculate L

$$\Delta = 16° 30'$$
$$= 16.5°$$
$$L = \frac{\pi}{180} R\Delta$$
$$= \frac{\pi}{180}(1100)(16.5)$$
$$= 316.777 \text{ m}$$

Calculate the PC station

PI - T = 10,000 - 159.492

= 9840.508 m

PC station is 9 + 840.508

Calculate the PT station

PC + L = 9840.508 + 316.777

= 10157.285

PT station is 10 + 157.285

Calculate the deflection for a 20 m interval

$$dc = \frac{L_c}{L}\left(\frac{\Delta}{2}\right)$$
$$= \frac{20}{316.777}\left(\frac{16.5}{2}\right)$$

= 0.520871149° 18' 08" = 0°31'15"

The first even station after the PC is 9 + 860.

Calculate the first deflection angle

Lc = 9860 - 9840.508

= 19.492 m

$$dc = \frac{L_c}{L}\left(\frac{\Delta}{2}\right)$$
$$= \frac{19.492}{316.777}\left(\frac{16.5}{2}\right)$$

= 0.507641022

= 0° 30' 28"

The last even station before the PT is 10 + 140

Calculate the last deflection angle

Lc = 10140 - 9840.508

= 299.492 m

$$dc = \frac{L_c}{L}\left(\frac{\Delta}{2}\right)$$
$$= \frac{299.492}{316.777}\left(\frac{16.5}{2}\right)$$

= 7.799837109

= 7° 47' 59"

Chord distances would now be calculated using
$C_1 = 2 \text{ R } \sin(dc)$

Example of Curve Data

Station	Point	dc	Curve Data
10+157.285	PT	8º15'00"	PI 10+000.000 D = 16°30' R = 1100 m L = 316.777 m T = 159.492 m
10+140		7º47'59"	
10+120		7º16'44"	
10+00		6º45'29"	
10+080		6º14'14"	
10+060		5º42'59"	
10+040		5º11'44"	
10+020		4º40'29"	
10+000		4º09'13"	
9+980		3º37'58"	
9+960		3º06'43"	
9+940		2º35'28"	
9+920		2º04'13"	
9+900		1º32'58"	
9+880		1º01'43"	
9+860		0º30'28"	
9+840.508	PC	0º00'00"	

The deflection at the PT must equal $\Delta/2$.

Running the Curve.

After completing the computations, it is necessary to establish the curve on the ground. When running the curve ahead on line (from PC to PT) the instrument is set on the PC, the plate set at zero and the telescope inverted for a sight on the back tangent. An alternative method would be to sight the PI without the telescope inverted if the PI has already been set and is visible.

Turn the deflection for the first even half station and accurately measure the proper distance to the desired station. Be sure to measure the chord distance and not the curve distance. The chord distance must be calculated. The backsight should be checked periodically to be certain that the instrument has not drifted.

The curve may be backed in from the PT by entering the total deflection and backing off to the PC. When the PC is sighted, zero should be read.

Radial Layout Method

With the advent of electronic surveying, the need to occupy control points such as PCs, POCs, and PTs no longer exists. Control points that are set off the roadway in the vicinity of the curve are used for layout.

There are computer and data collector programs that calculate angles and distances from control points off the curve for setting points on the curve.

Coordinates of the curve alignment (such as 10 m stationing) must be input into the Data Collector or computer with the off-the-curve control point coordinates.

The program then calculates the angles and distances from control points to layout the curve.

Specific information about this procedure may be found in the SDR 33 reference manual under the “Roading”chapter.

Also, the design engineering software has commands and procedures to generate radial layout data.

Instrument Set At POC

Assuming that the first part of a curve has been located by deflections from the PC, if the next part of the curve is not visible from the PC it must be located by deflections from some point on the curve, usually at a full station.

The instrument can be set on a point from which the PC can be backsighted (Methods A and B) or on any point from which some intermediate POC can be backsighted (Method C).

Method A uses deflection angles turned from the auxiliary tangent at the POC being occupied. Methods B (preferred) and C (for remote locations) use the original calculated (book) deflections turned from the extension of the chord from the PC to the occupied POC.

Method A

- Set the scale at the deflection angle for the point being occupied, but to the “wrong side”.
- Backsight on the PC with the scope inverted
- 0° is now an auxiliary tangent.
- Turn deflection angles for the forward points based on their distance from the occupied POC and therefore turned from the auxiliary tangent.

Method B

- Set the scale to 0°
- Backsight on the PC with the scope inverted.
- 0° is now the extension of the chord from the PC to the occupied POC.
- Turn deflection angles to the forward points using the original calculated (book) deflection angles.

Method C

- Set the scale at the deflection angle for an intermediate point being sighted (POC_1): df_1
- Backsight on the intermediate POC_1 with the scope inverted.
- 0° is now the extension of the chord from the PC to the occupied POC.
- Turn deflection angles to the forward points using the original calculated (book) deflection angles.

Offset Curves

Frequently it is necessary to locate outer and inner concentric curves, such as property lines, curb lines and offset curves for reference during construction. The full lengths of these offset curves are also desirable. Curve data can be calculated using the adjusted radius or by proportioning the center line data.

The subscripts "o" for outside and "i" for inside are commonly used to identify elements on offset curves.

For example, the length of an inside curve would be $L_i = \frac{\pi}{180} R_i \Delta$; R_i being a shorter radius than the center line radius R. Or, by proportion, $L_i = (R_i/R)\ L$.

It is convenient to note that the difference in arc length L between the offset curve and the center line curve, for the same internal angle Δ, where w is the offset distance, is $(2\pi\Delta / 360°)\ w$.

Degree of Curvature

WSDOT does not use degree of curvature with the metric system but a knowledge of the convention is necessary when investigating existing facilities.

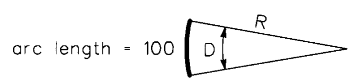
The two common definitions of degree of curvature (D) are the arc definition used in highway work before metrication and the chord definition used by some counties and in railroad work.

By the arc definition, a D degree curve, has an arc of 100 feet (not meters) resulting in an internal angle of D degrees. (So, the stationing and angles are known and the chords remain to be calculated.)

By the chord definition, a D degree curve has a chord of 100 feet (not meters) resulting in an internal angle of D degrees. (So, the chords and angles are known and the arc stations would remain to be calculated.)

In terms of radius, a 1° curve by the arc definition would have a radius of 5729.578 feet. And by the chord definition, its radius is 5729.65 feet. In the days of sliderules, a radius of 5,730 feet might have been used as the formula $R = \frac{5730}{D}$ in the Field Tables for Engineers, Spirals, 1957.

Degree of Curvature for Various Lengths of Radii

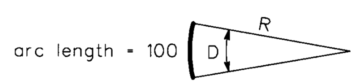


Exact for Arc Definition

$$D = \frac{100\left(\frac{180}{\pi}\right)}{R} = \frac{18000}{\pi R}$$

Where D is Degree of Curvature

Length of Radii for Various Degrees of Curvature



$$R = \frac{100\left(\frac{180}{\pi}\right)}{D} = \frac{18000}{\pi D}$$

Where R is Radius Length

Degree of Curvature (Highway)
Figure 10–3

Field Record

When not using a data collector it is necessary to hand-write field notes in the traditional manner.

The field notes for curves are kept on transit note sheets. These are available on regular or “rite in rain” paper.

The left page is for stations, curve data, deflections, and other such data. The right page is for ties to curve points, descriptions of points set and any pertinent drawings that may make clear to others just what was done in the field. This is very important as the work may have to be reproduced years later.

It is useful to show the location of points relative to permanent objects. Note what the point is (spike, hub & tack, etc.), and how it is referenced.

Make notes that are neat and accurate. Title and index the first page of each operation. On the first page of each day’s work, show the date, crew, weather, and instrument by serial number. Number and date every page. Notes written with the book turned are written with the right edge of the book toward the writer.

Do not use “scratch” notes intending to put them in the book later.

Field records are an important aspect in any survey. A well executed surveying job is worthless unless it is well documented. Take the time necessary, in the field, to create good records of what you have done.

P:HSM10

11 Spiral Curves

WSDOT does not use spiral curves on new highway design but a knowledge of spiral curves is necessary when an existing highway alignment contains spiral curves. Since the known design parameters are in English units, it is simplest to use the English formula to calculate desired dimensions and then convert the results to metric units.

A spiral curve is for the transition of a vehicle traveling at a sustained speed from a straight tangent to a circular curve. It is an attempt to approximate the path followed by a vehicle's wheels from when the operator begins to turn his steering wheel until he has reached the maximum degree of curvature at the circular portion of the curve.

Spiral curves are divided into an entering spiral transition, a circular curve, and an exiting spiral transition. In most cases the entering and exiting spiral will be equal. The major difference between a spiral and a circular curve is that the change of direction varies as the square of the length for a spiral rather than as the first power of the length for a circular curve. The degree of curvature on a spiral increases directly as the distance increases along the spiral curve from the tangent. The degree of curvature at any point in the spiral is the same as the degree of curvature of a circular curve having the same radius. A spiral curve will be tangent to a circular curve at the point where they share the same radius.

Spiral Curve Elements

For circular curve elements see Chapter 10.

a — Rate of change in the degree of curve of a spiral per 100 feet of length which equals the degree of curve on a spiral at a point 100' (one station) from the TS (or ST).
$a = D/L_S$ or $a = D_S/L_S'$

CΔ — Central angle of circular curve between connecting spiral curves.
$C\Delta = \Delta - 2DE$ for equal spirals
$C\Delta = \Delta - (DE^1 + DE^2)$ for unequal spirals

CS — Point of change from circular curve to spiral.
$CS = ST - (L_S \times 100)$

C_S — Total chord length for a spiral curve from its beginning (TS or ST) to its end (SC or CS).
$C_S = 100'\, L_S - 0.000338\, a^2(L_S)^5$;
also $= \sqrt{x^2 + y^2}$

C_S' — Chord length to any point on a spiral from TS or ST.
$C_S' = 100'\, L_S' - 0.000338\, a^2(L_S')^5$; also $= \sqrt{(x')^2 + (y')^2}$

Δ — (delta) Total intersection or “central” angle between back and forward tangents.

D — When used in reference to a spiral, D indicates the maximum degree of curvature of the spiral which is at SC or CS.
$D = aL_S$

DE — Deviation angle of spiral measured from back tangent (or forward tangent) to tangent through the spiral at its maximum degree.
$DE = [a(L_S)^2]/2$; also $= DL_S/2$; also $= DF + DH$

de — Deviation angle of spiral measured from back tangent (or forward tangent) to tangent through any point on spiral.
$de = [a(L_S')^2]/2$; also $= D_S L_S'/2$; also $= df + dh$

df — Deflection angle to any point on spiral measured from tangent at TS (or ST).
$df = a(L_S')^2/6 - dfk$ in degrees

DF — Deflection angle for full spiral measured from tangent at TS (or ST) to SC (or CS) respectively.
$DF = a(L_S)^2/6 - DFk$; also $D_S L_S/6 - DFk$;
also $= DE/3 - DFk$

dfk — A correction, in minutes, to be applied to the equation for df when the angle de is 15° and over.
$dfk = 0.000053(DE)^3$

DFk — A correction, in minutes, to be applied to the equation for DF when the angle DE is 15° and over.
$DFk = 0.000053(DE)^3$

dh — Deflection angle required to establish tangent to any point on the spiral measured between tangent to the spiral at that point and the beginning of the spiral (TS or ST).
$dh = de - df$

DH — Deflection angle required to establish tangent to the spiral at its maximum degree when instrument is sighted on the beginning of the spiral (TS or ST).
$DH = DE - DF$

dr — Deflection angles required to be computed for each intermediate setup on a spiral to locate other points on the spiral. When sighting on TS or ST to establish tangent to the spiral, dr = dh.

$$dr = df + \frac{D_S L_S'}{6}$$

D_S — Degree of curve at any point on spiral.
$D_S = aL_S'$

L_S — Total length of a spiral curve measured along its arc in stations of 100'.
$L_S = D/a$

L_S' — Length of spiral curve from T.S. (or S.T.) to any point on spiral measured along its arc in stations of 100'
$L_S' = D_S/a$

PCO — Point where circular curve, if extended around its center, has a tangent that is parallel to the spiral semi-tangent.

POS — Point on spiral.

Q Offset distance perpendicular to the forward and back tangents that the circular curve is moved to accommodate the spiral transitions.
$Q = (0.0727a)(L_S)^3 - (0.0000002a^3)(L_S)^7$
(See Note 1)

R Radius of circular curve in feet (minimum length of R_S for spiralled curve).

R_S Radius in feet at any point on spiral curve.
$R_S = 5730/D_S$

SC Point of change from spiral to circular curve.
$SC = TS + (L_S \times 100)$

ST Point of change from spiral to forward tangent.

t Distance along semi-tangent of a spiral curve from TS (or ST) to perpendicular offset through PCO in feet.
$t = 50L_S - (0.000127a^2)(L_S)^5$

T Distance along semi-tangent from the point of intersection of the back and forward tangents to the origin of curvature from that tangent.
$T = t + (R + Q) \tan \Delta/2$

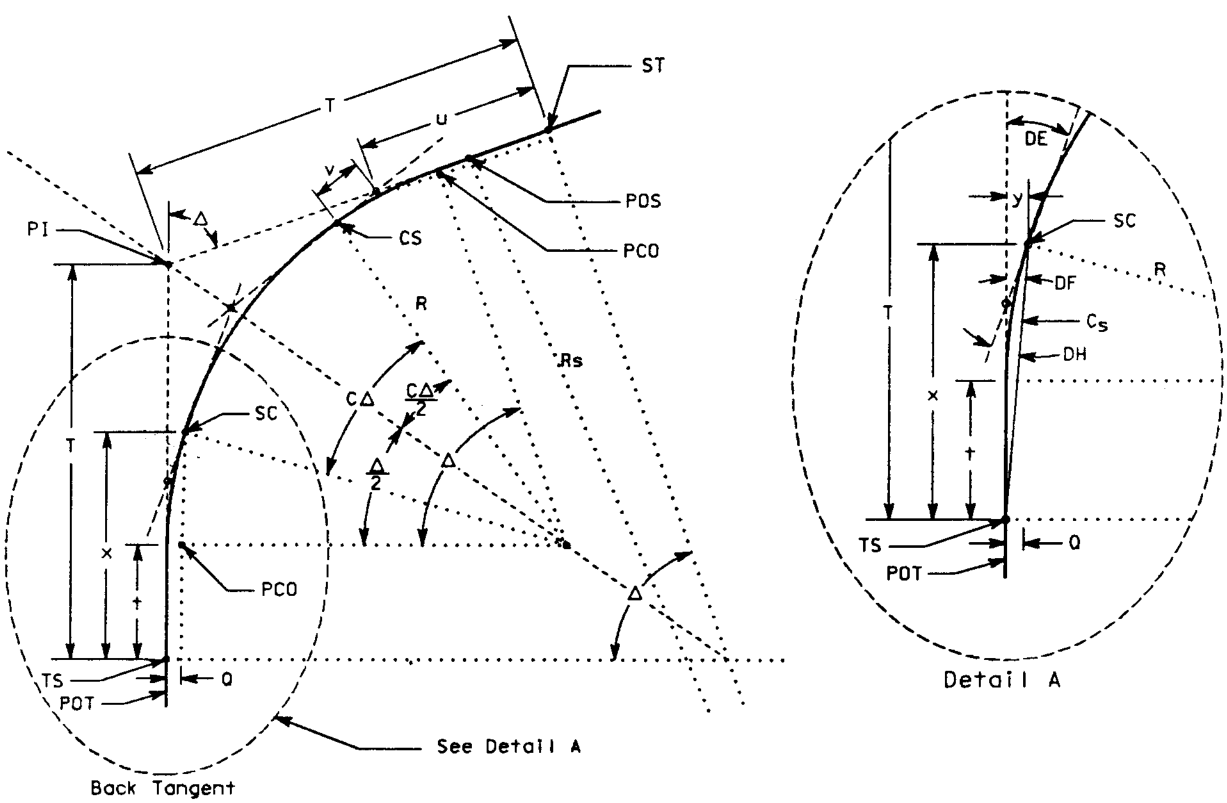
TS Point of change from back tangent to spiral.
$TS = PI - T$

x Distance along semi-tangent of a spiral curve from TS (or ST) to perpendicular offset to the end of spiral in feet.
$x = 100L_S - [(0.000762a^2)(L_S)^5 + (0.0000000027a^4)(L_S)^9]$ (See Note 1)

x' Distance along semi-tangent of a spiral curve from TS (or ST) to perpendicular offset to any point on spiral in feet.
$x' = 100L_S' - [(0.000762a^2)(L_S')^5 + (0.0000000027a^4)(L_S')^9]$ (See Note 1)

y Offset distance from the semi-tangent to the SC (or CS) measured perpendicular to the semi-tangent in feet.
$y = (0.291a)(L_S)^3 - (0.00000158a^3)(L_S)^7$ (See Note 1)

y' Offset distance from the semi-tangent to any point on the spiral measured perpendicular to the semi-tangent.
$y' = (0.291a)(L_S')^3 - (0.00000158a^3)(L_S')^7$
(See Note 1)



Spiral Curve Elements
Figure 11–1

SPIRAL TABLES a = 1⅔—Continued 1° in 60 ft.

Ls′	Ds	de	df	Q	R + Q	t	x	y	Cs′
Stations	Degrees and Minutes	Deg. & Min.	Deg. & Min.	Feet	Feet	Feet	Feet	Feet	Feet
3.1	5°-10′	8°-00.50′	2°-40.1394′	3.607	1112.639	154.900	309.393	14.428	309.731
3.2	5°-20′	8°-32.00′	2°-50.6337′	3.967	1078.342	159.882	319.289	15.867	319.685
3.3	5°-30′	9°-04.50′	3°-01.4604′	4.350	1046.168	164.862	329.170	17.398	329.633
3.4	5°-40′	9°-38.00′	3°-12.6193′	4.757	1015.933	169.840	339.037	19.024	339.573
3.5	5°-50′	10°-12.50′	3°-24.1102′	5.189	987.474	174.815	348.886	20.747	349.507
3.6	6°-00′	10°-48.00′	3°-35.9332′	5.645	960.645	179.787	358.718	22.570	359.432
3.7	6°-10′	11°-24.50′	3°-48.0879′	6.128	935.317	184.756	368.529	24.497	369.349
3.8	6°-20′	12°-02.00′	4°-00.5744′	6.638	911.374	189.721	378.319	26.529	379.256
3.9	6°-30′	12°-40.50′	4°-13.3921′	7.174	888.712	194.682	388.085	28.669	389.153
4.0	6°-40′	13°-20.00′	4°-26.5410′	7.739	867.239	199.639	397.827	30.920	399.039
4.1	6°-50′	14°-00.50′	4°-40.0209′	8.332	846.868	204.592	407.540	33.284	408.912
4.2	7°-00′	14°-42.00′	4°-53.8316′	8.955	827.526	209.539	417.225	35.764	418.773
4.3	7°-10′	15°-24.50′	5°-07.9728′	9.608	809.142	214.482	426.877	38.362	428.620
4.4	7°-20′	16°-08.00′	5°-22.4440′	10.291	791.654	219.419	436.496	41.080	438.452
4.5	7°-30′	16°-52.50′	5°-37.2453′	11.006	775.006	224.350	446.078	43.922	448.267
4.6	7°-40′	17°-38.00′	5°-52.3760′	11.753	759.144	229.274	455.621	46.889	458.066
4.7	7°-50′	18°-24.50′	6°-07.8361′	12.532	744.021	234.191	465.122	49.983	467.847
4.8	8°-00′	19°-12.00′	6°-23.6249′	13.345	729.595	239.102	474.578	53.207	477.608
4.9	8°-10′	20°-00.50′	6°-39.7421′	14.192	715.824	244.004	483.987	56.563	487.348
5.0	8°-20′	20°-50.00′	6°-56.1874′	15.073	702.673	248.898	493.344	60.053	497.066
5.1	8°-30′	21°-40.50′	7°-12.9603′	15.989	690.106	253.783	502.648	63.679	506.761
5.2	8°-40′	22°-32.00′	7°-30.0603′	16.941	678.094	258.659	511.894	67.442	516.430
5.3	8°-50′	23°-24.50′	7°-47.4868′	17.930	666.609	263.525	521.079	71.346	526.074
5.4	9°-00′	24°-18.00′	8°-05.2395′	18.955	655.621	268.381	530.199	75.391	535.689
5.5	9°-10′	25°-12.50′	8°-23.3176′	20.018	645.108	273.225	539.251	79.578	545.275
5.6	9°-20′	26°-08.00′	8°-41.7208′	21.118	635.046	278.058	548.230	83.911	554.829
5.7	9°-30′	27°-04.50′	9°-00.4481′	22.258	625.415	282.878	557.131	88.389	564.351
5.8	9°-40′	28°-02.00′	9°-19.4990′	23.426	616.194	287.685	565.952	93.015	573.838
5.9	9°-50′	29°-00.50′	9°-38.8729′	24.654	607.365	292.478	574.686	97.789	583.288
6.0	10°-00′	30°-00.00′	9°-58.5690′	25.912	598.912	297.257	583.330	102.713	592.699

Sample Spiral Table
Figure 11–2

Note 1

The last term in the equation may be omitted when the value of DE is 15° or less.

An example of a circular curve with equal spirals is calculated.

Given:

$$\Delta = 100°\ 00'$$
$$PI = \text{Station } 120 + 10.54$$
$$D = 6°\ 00'$$
$$L_S = 3.6 \text{ Stations}$$

Determine the information necessary to establish the control points for the curve.

First, determine a, the rate of change in the degree of curvature of the spiral per 100 ft of curve.

$$\begin{aligned} a &= D/L_S \\ &= 6°/3.6 \\ &= 1\ 2/3 \text{ degrees/station} \end{aligned}$$

Now the spiral tables can be used to find information used to establish the control points for the curve.

From the *Field Tables for Engineers, Spirals 1984,* page 57, (a=1 2/3) the following information is found.

de	= 10° 48'	t	= 179.787'
df	= 3° 36'	x	= 358.718'
Q	= 5.645'	y	= 22.570'
R + Q	= 960.645'	C_S	= 359.432'

Or, if spiral tables are not available, the calculations go as follows.

First the angles DE, DF, DH and CΔ are calculated.

$$\begin{aligned} DE &= DL_S/2 \\ &= (6)(3.6)/2 \\ &= 10.8° \\ &= 10°\ 48' \end{aligned}$$

$$
\begin{aligned}
DF &= DE/3 - DFk \\
&\quad (DFk = 0 \text{ because } D < 15°) \\
&= (10° 48')/3 - 0 \\
&= 3° 36'
\end{aligned}
$$

$$
\begin{aligned}
DH &= DE - DF \\
&= 10° 48' - 3° 36' \\
&= 7° 12'
\end{aligned}
$$

$$
\begin{aligned}
C\Delta &= \Delta - 2DE \\
&= 100° - (2)(10° 48') \\
&= 78° 24'
\end{aligned}
$$

Once CΔ is known, the length of the circular curve can be found.

$$
\begin{aligned}
L &= C\Delta/D \\
&= (78° 24')/6 \\
&= 13.0667 \text{ stations}
\end{aligned}
$$

The radius for the circular curve is calculated using the formula in Field Tables for Engineers Spirals 1984, page 8.

$$
\begin{aligned}
R &= 5730/D \\
&= 5730/6 \\
&= 955.0'
\end{aligned}
$$

To find the stations, first calculate T. Since T = t + (R + Q) tan D/2, it is necessary to first find Q and t.

$$
\begin{aligned}
Q &= (0.0727a)(L_S)^3 \\
&= (0.0727)(5/3)(3.6)^3 \\
&= 5.6532'
\end{aligned}
$$

$$
\begin{aligned}
t &= 50\ L_S - (0.000127a^2)(L_S)^5 \\
&= 50(3.6) - (0.000127)(1\ 2/3)^2(3.6)^5 \\
&= 180 - 0.2133 \\
&= 179.7867'
\end{aligned}
$$

Now find T.

$$
\begin{aligned}
T &= t + (R + Q)\tan \Delta/2 \\
&= 179.7867 + (955 + 5.6532)\tan 100/2 \\
&= 1324.65'
\end{aligned}
$$

Stationing can now be determined for the control points.

PI = station 120 + 10.54 is given

$$
\begin{aligned}
TS &= PI - T \\
&= 12010.54 - 1324.65 \\
&= \text{station } 106 + 85.89
\end{aligned}
$$

$$
\begin{aligned}
SC &= TS + (L_S \times 100) \\
&= 106 + 85.89 + 360 \\
&= \text{station } 110 + 45.89
\end{aligned}
$$

$$
\begin{aligned}
CS &= SC + (L\text{x}100) \\
&= 110{+}45.89 + 1306.67 \\
&= \text{station } 123{+}52.56
\end{aligned}
$$

$$
\begin{aligned}
ST &= S + (L_S\text{x}100) \\
&= 360 \\
&= \text{station } 127{+}12.56
\end{aligned}
$$

Spiral Deflections

For the deflections the spirals are divided into equal arcs. In the example above, where L_S = 360', the spirals may be divided into nine equal arcs of 40 feet each. (A railroad would have used chords.) A curve with equal spirals uses the same arcs and deflections at either end.

Since the deflection varies as the square of the distance (L_S'), the deflection angle df = a(L_S')^2/6 in degrees or 10a(L_S')^2 in minutes at any point on the spiral. The rate of change of curvature was calculated first and is 1⅔ or, to calculate more simply, a = ⁵⁄₃ .

The deflections for the previous example are, therefore:

Station	**Deflection**
106+85.89 TS	0°
107+25.89	$(5/3)(0.4)^2/6 = 0.044° = 0° 02' 40''$
107+65.89	$(5/3)(0.8)^2/6 = 0.177° = 0° 10' 40''$
108+05.89	$(5/3)(1.2)^2/6 = 0.399° = 0° 24' 00''$

This process is continued until the S.C. is reached at station 110+45.89 where L_S' is 3.6. The deflections are the same for the spiral at the other end of the curve. The circular curve is run in the same manner as a circular curve without spirals on either end.

For further information concerning spiral curves consult the *Field Tables for Engineers, Spirals 1984.*

Metric Considerations

Since spirals are not used in new highway design, there will be no new spirals developed with metric design parameters. Any spirals encountered will have design parameters based on English Units (feet and 100' stations). To develop spiral stations in metric units for an existing alignment; develop spiral control points and deflection angles using English design parameters and methods described in the previous section — then convert English units to metric units for project work. (Deflection angles will remain the same.)

Example:

Develop spiral control points and deflections given the following information:

Delta = 100º00'
PI = Station 3 + 660.820 (m)
D = 6º00'
L(s) = 3.6 Stations (ft)

1. Convert metric units to English units.

   Using a conversion factor of $\frac{39.37}{12}$ feet per meter for the PI at station 3+660.820m.

   (3660.820 m)*($\frac{39.37}{12}$ ft/1 m) = 12010.54 ft

   Thus the PI station is 120+10.54 (ft)

2. Develop spiral control points and deflection angles using English design parameters and methods described in the previous section.

   Control point stationing and deflection angles were calculated in the previous section using English units.

3. Convert English units to metric units (deflection angles will remain the same).

Using the conversion factor of $\frac{39.37}{12}$ meters per foot:

Control points:

	Station (ft)	conversion factor	Station (m)
PI	120+10.54	$\frac{39.37}{12}$	3+660.820
TS	106+85.97	$\frac{39.37}{12}$	3+257.097
SC	110+45.97	$\frac{39.37}{12}$	3+366.818
CS	123+52.64	$\frac{39.37}{12}$	3+765.042
ST	127+12.64	$\frac{39.37}{12}$	3+874.820

Deflections:

Station (ft)	conversion factor	Station (m)	Deflection
106+85.97 TS	$\frac{39.37}{12}$	3+257.090	0º 00' 00"
107+25.97	$\frac{39.37}{12}$	3+269.282	0º 02' 40"
107+65.97	$\frac{39.37}{12}$	3+281.474	0º 10' 40"
108+05.97	$\frac{39.37}{12}$	3+293.666	0º 24' 00"

Any additional layout information needed to stake out curves (such as arc length or chord length) can be converted to metric units in a similar manner.

Field Procedures

One method for running a spiral is by occupying the TS (or ST). Set zero in the instrument and backsight a point on tangent or foresight a point on semi-tangent. Turn the calculated deflection and chain from station to station along the spiral.

Another method is to set out coordinate values for selected stations calculated with coordinate geometry software.

Offset Curves

It may be necessary to locate outer or inner concentric curves such as lane edges and offset curves for reference during construction. Spiral curve data can be calculated using the offset width (w) and the deflection angles for the segment: one measured from the forward tangent to the POS_1 at the beginning of the segment (df_1), and the other measured to the POS_2 at the end of the segment (df_2).

To sight along a line tangent to the spiral at any point (POS_2) on a spiral:

1. Occupy the TS.
2. Sight an intermediate point (POS_1) for the deflection angle df_1.
3. Sight POS_2 for angle df_2.
4. Occupy POS_2 and sight POS_1.
5. Turn an angle = $2(df_2 - df_1)$ away from the TS to sight the tangent.

To sight radially at POS_2 follow steps 1 through 4 above and then turn 90° in either direction.

The length of a spiral curve segment of an offset curve is the center line segment plus (outside) or minus (inside curve) the amount $3[\sin(df_2 - df_1)]w$.

See Chapter 10, Circular Curves.

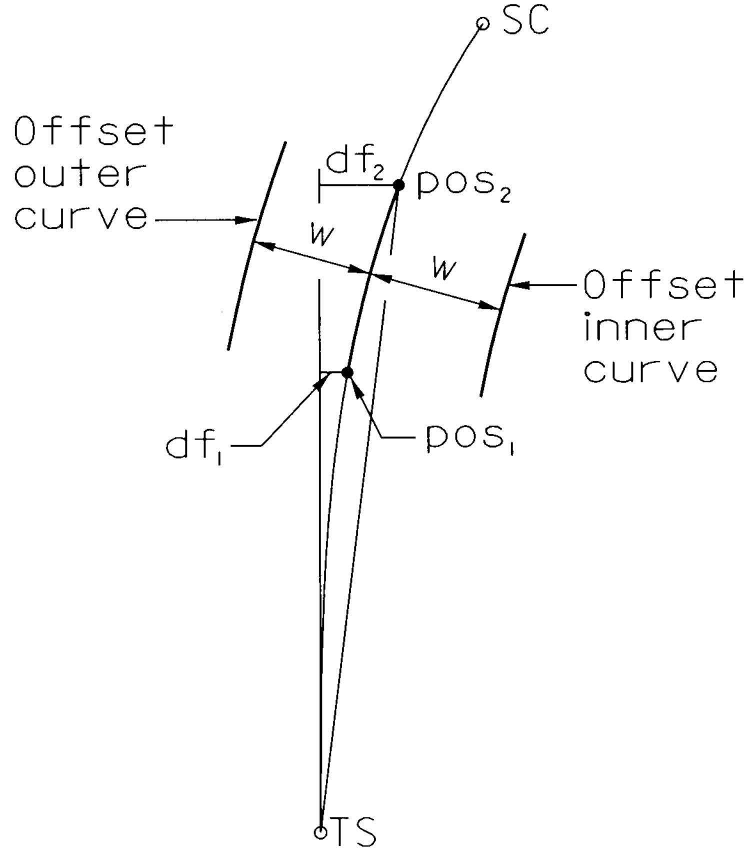


Figure 11–3

P:HSM11

12
Bridge Site Surveys

Coordinate with the Bridge Designer at the Bridge and Structures Office, Bridge Projects Unit regarding the area needed for the bridge site map.

Stake the control center line. Collect precise DTM data covering proposed right of way plus at least one additional shot beyond right of way and 40 meters beyond the proposed bridge. Generate an accurate contour map from the DTM data.

Contact utility owners to have their underground facilities marked. These include sewers, water lines, gas lines, culverts, power and telecommunication lines. If possible, note the elevations of these facilities, along with pipe or conduit size. Tie down each utility location reference mark using the appropriate codes and notes.

In addition to the above, the following structures require the site-specific data noted:

Railroad Grade Separations

For highway-railroad grade separations, locate all tracks within 200 m of the center line of the proposed highway and include the following data:

- Points along spirals and circular curves.
- Distance from center to center of rails and distance from track center line to railroad right-of-way line.
- Location of switch points, (track crossings), and other track appurtenances.
- Profile (accurate to 0.005 m) taken atop each rail for 200 m each side of the intersection.
- Equation in elevations between the railroad datum and highway datum.
- Elevation of the top of each rail at its intersection with the roadway center line.

Highway Grade Separations

For highway-highway grade separations, the intersection stations must be noted, along with the angle of intersection. Locate the intersection on the plan layout and include the following data regarding the existing highway:

- Alignment with pertinent horizontal curve data.
- Width of traveled roadway, shoulders, location and depth of ditches.
- Profile taken along center lines and each edge of traveled way (on fog line) for a distance of at least 200 m each side of the intersection.

P:HSM12

13

Vertical Alignment and Superelevation

Profile Grade

Grade is the rate of change in vertical elevation per unit of horizontal length. This rate is expressed in percent. For example, a 1 percent grade means a rise or fall of 1 unit in 100 units of horizontal distance (rise over run).

To determine the grade between two points on a line, divide the difference in elevation by the distance between the two points and multiply by 100 so the result will be in percent.

Example (metric)

	Station		Elevation
	24+160		21.00 m
	25+670		84.29 m
distance	1+510	difference in elevation	63.29 m

The grade in % is $\dfrac{63.29\text{ m}}{1510\text{ m}} \times 100 = 4.19\%$

Since the elevation ahead on line is higher than the elevation of the beginning station it is a plus grade. If the ahead elevation were lower, it would be a minus grade.

This **profile grade** describes the vertical alignment of the roadway and is shown on the profile sheets of the contract plans.

A finished roadway is not a flat plane. Instead, the roadway is sloped slightly to the sides to allow water to run off. This requires that a definite point on the cross section be chosen to "carry" the grade. The lateral location of profile grade is shown in the roadway sections portion of the contract plans.

On construction, it is important to study the profile sheets and roadway sections and know where every change in the profile grade occurs. Serious staking errors can be made in the field by overlooking a lateral shift in the location of the profile grade.

Vertical Curves

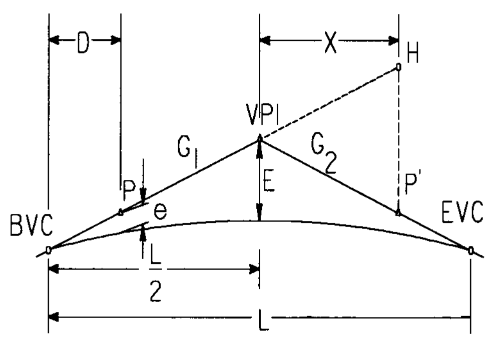
When a change in grade of more than about 0.5 percent occurs at a VPI, a vertical curve is required. The vertical curve lengths are determined by criteria found in the *Design Manual*. Once the grades are established and the lengths of the curves are chosen, the vertical offsets of the curves can be computed. A vertical curve is a parabolic curve. When the grades form a peak or hill at the VPI, the curve is known as a **crest vertical** or **summit vertical** (Figure 13-1). When the grades form a valley or dip at the VPI the curve is known as a **sag vertical** (Figure 13-2). When l_1 does not equal l_2 as shown in Figure 13-3 the curve is nonsymmetrical.

The vertical curve is computed by figuring offsets from the tangent grades. Subtract the offsets from the tangent grade elevations for crest verticals and add the offsets to the tangent grade elevations for sag verticals.

The following nomenclature and formulas are from the *Highway Engineering Field Formulas,* M 22-24.

Nomenclature For Vertical Curves

G_1 & G_2	Tangent grade in %
A	The absolute of the algebraic Difference in grades in %
BVC	Beginning of Vertical Curve
EVC	End of Vertical Curve
VPI	Vertical Point of Intersection
L	Length of vertical curve
D	Horizontal Distance to any point on the curve from BVC or EVC
E	Vertical distance from VPI to curve
e	Vertical distance from any point on the curve to the tangent grade
K	Distance required to achieve a 1% change in grade
L_1	Length of a vertical curve which will pass through a given point
D_0	Distance from the BVC to the lowest or highest point on curve
X	Horizontal distance from P' to V
H	A point on tangent grade G_1 to vertical position of point P'
P and P'	Points on tangent grades



Crest Vertical Curve
Figure 13–1

$$e = \frac{AD^2}{200L}$$

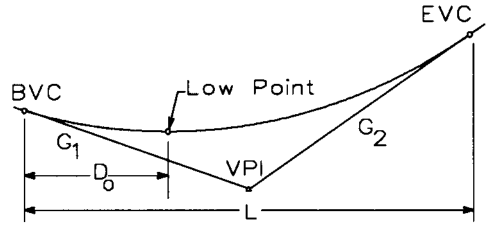
$$L_1 = \frac{2(AX + 200e + 20\sqrt{AXe + 100e^2})}{A}$$

$$D_0 = |G_1|\frac{L}{A}$$

$$X = \frac{100(ElevH - ElevP')}{A}$$

$$K = \frac{L}{A}$$

Equations for Crest Vertical Curve



Sag Vertical Curve
Figure13–2

$$E = \frac{AL}{800}$$

$$E = \frac{1}{2}\left(\frac{Elev.BVC + Elev.EVC}{2} - Elev.VPI\right)$$

$$e = \frac{4ED^2}{L^2}$$

Notes: All equations use units of length (not stations or increments)

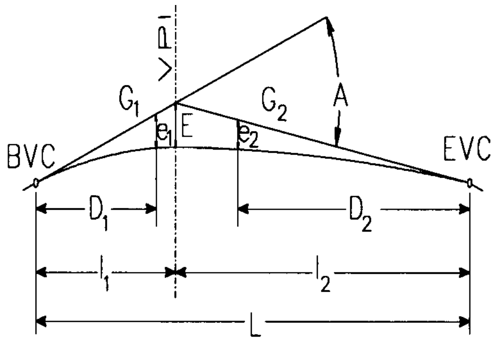
The variable **A** is expressed as an absolute in (%) percent

Example: If G_1 = +4% and G_2 = -2%
Then **A**=6

Equations for Sag Vertical Curve

Nomenclature For Nonsymmetrical Vertical Curves

G_1 & G_2	Tangent grades in %
A	The absolute of the algebraic difference in grades in Percent
BVC	Beginning of Vertical Curve
EVC	End of Vertical Curve
VPI	Vertical Point of Intersection
l_1	Length of first section of vertical curve
l_2	Length of second section of vertical curve
L	Length of vertical curve
D_1	Horizontal Distance to any point on the curve from BVC towards the VPI
D_2	Horizontal Distance to any point on the curve from EVC towards the VPI
e_1	Vertical distance from any point on the curve to the tangent grade between BVC and VPI
e_2	Vertical distance from any point on the curve to the tangent grade between EVC and VPI
E	Vertical distance from VPI to curve



Nonsymmetrical Vertical Curve
Figure 13–3

$$A = \left|(G_2) - (G_1)\right|$$

$$L = l_1 + l_2$$

$$E = \frac{l_1 l_2}{200(l_1 + l_2)} A$$

$$e_1 = m\left\{\frac{D_1}{l_1}\right\}^2$$

$$e_2 = m\left\{\frac{D_2}{l_2}\right\}^2$$

Equations for Nonsymmetrical Vertical Curve

Superelevation

On horizontal curves, the roadway is tilted so that the edge of the pavement at the outside of the curve is higher than the edge of the pavement at the inside of the curve. This is called **superelevation** and is done to counteract the centrifugal force which tends to push the vehicle off the roadway at the outside of the curve.

The rate of superelevation is a function of the radius of the curve and the design speed. As the radius shortens for a given design speed or as the design speed increases for a given radius, the **super rate** must increase to keep the vehicle on the road. The maximum super rate is 0.10. For further information see Chapter 640 of the *Design Manual*.

Going from a tangent section which has a **normal crown** to a curve section which has **full super**, there is a gradual change in the rate of superelevation called a **transition**. Approximately three fourths of the transition is in the tangent section of the roadway and one fourth is in the curve. See the *Design Manual* for superelevation transition designs. The edge of the pavement that is on the outside of the curve begins to rise in relation to the center line (or reference point). The edge rises until the roadway is on one (sloping) plane from edge to edge. This is called **crown slope** and is usually 0.02. The entire roadway then rotates about the **pivot point** until it reaches its maximum or **full super**. Exiting a curve, the process is simply reversed, going from full super to crown slope to normal crown. See Figure 13-4.

The survey crew should not have to design supers but may have to compute grades for a station or offset on a section in transition. The contract plans will contain superelevation diagrams on the same sheet as the roadway profiles. Figure 13-5 shows a super diagram for the roadway shown in Figure 13-4.

The contract plans and/or the roadway elevation listing (grade sheets) show the stations of the beginning of the transition, crown slope, and full super.

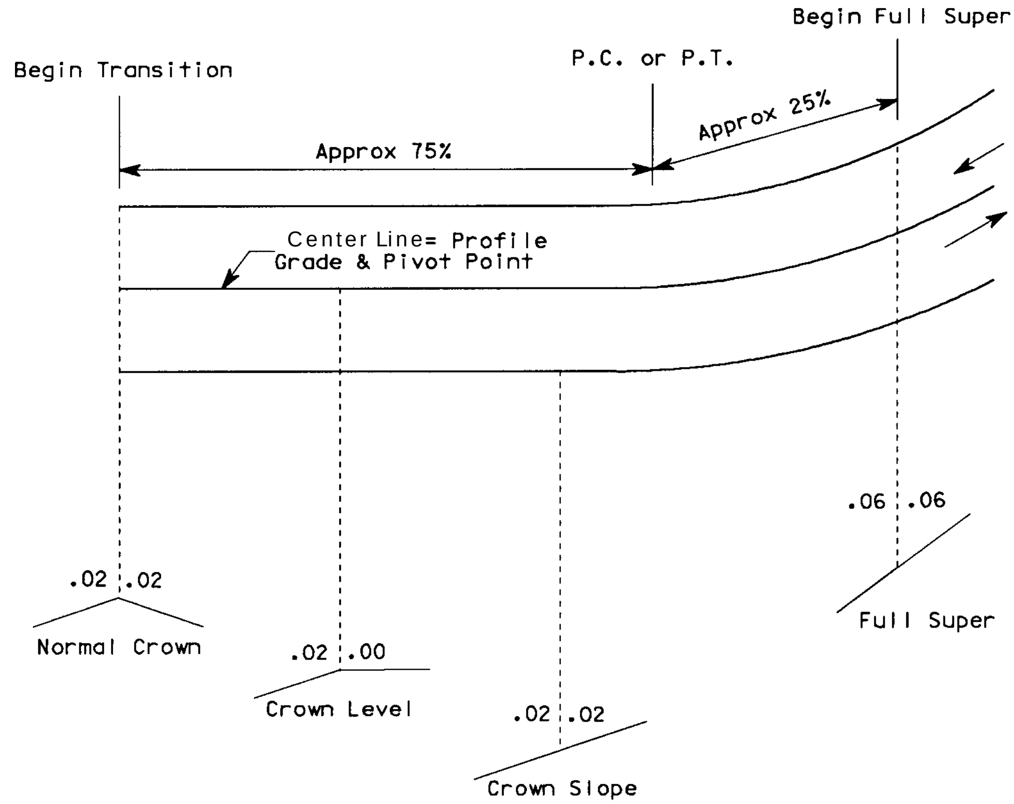
To find the super rate for any station in a transition:

1. Subtract the begin transition station from the end transition station.
2. Subtract the desired station from the end transition station.
3. Subtract, algebraically, the crown slope from the full super rate.
4. Divide the super difference (#3) by the station difference (#1).
5. Multiply the desired station difference (#2) by the rate of change (#4).
6. Subtract #5 from the full super rate which gives the super rate for the station in question.

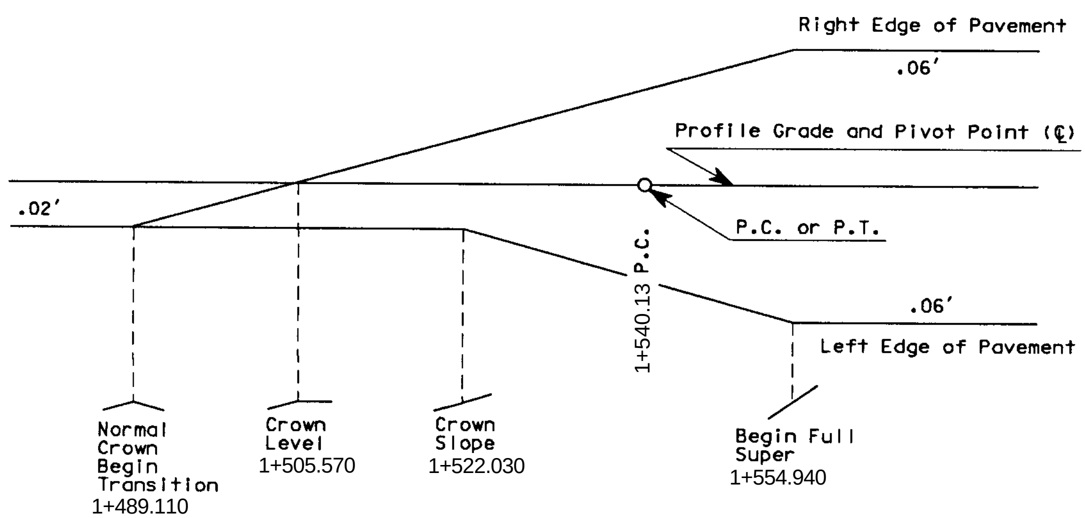
Example

Find the super rate at station 1+535 on the right edge of the pavement using Figure 13-5.

1. 1+554.940 - 1+489.110 = 65.830 m
2. 1+554.940 - 1+5350 = 19.940 m
3. 0.06 - (-0.02) = 0.08 m/m
4. 0.08/65.830 = 0.0012150 m/m/m = rate of change
5. 0.001215 x 19.940 = 0.02423 m/m
6. 0.06 - 0.02423 = 0.03577 m/m



Superelevation for Two-Lane Highway
Figure 13–4



Superelevation Diagram for Two-Lane Highway
Figure 13–5

P:HSM13

14 Earthwork and Surfacing

This chapter begins the first in a series on construction staking and layout. In performing this type of surveying preparation is a major part of the operation. Study the contract plans, special provisions, standard plans, standard specs, the plan quantities, *Construction Manual*, and the contractor's proposal. Take appropriate measures to protect existing monuments.

Features may be staked directly from the highway center line by station and offset, or they may be staked from some other control point which may be more convenient. Depending on the type of work to be performed by the contractor, stakes may be set to mark the actual work, such as clearing limits or they may be offset in staking pipe.

After completing a staking operation look at the stakes to be sure that they are in a uniform line if they should be. Review cuts or fills recorded on the stakes to ensure that they agree with the notes.

In addition to "Field Note Records" (which are kept as documentation for payments to the contractor), computer printouts, and cross section notes, keep a field book to record alignment, references, control points, slope stake data, drainage computations, and other information.

As the construction work progresses, the survey crew is involved in almost every phase.

First, stakes are set for the clearing and grubbing phase. The contractor removes buildings, foundations, underground utilities, trees, bushes, sod, stumps, and other obstacles to highway construction work. The survey crew may take cross-section or DTM data for calculating pay quantities and set slope stakes so the contractor can do the earthwork.

After completion of the earthwork, the survey crew sets grade control stakes, blue tops for the final subgrade work and for each layer of surfacing material, red tops for base course, and yellow tops for top course.

Clearing and Grubbing

Use the center line of the roadway as the reference line for setting clearing stakes. Set stakes (lath) at least 1.2 m long marked "Clearing," at the proper offset, marking the limits of the area to be cleared. Set the stakes at intervals as shown in Figure 14-6. Where slope treatment is to be provided, clearing normally is staked 3.0 m beyond the limits of the slope treatment with 1.5 m minimum. Do not set grading stakes until clearing and grubbing work in a given area is completed. Refer to the Standard Specifications, Section 2-01.

Measuring clearing at interchanges can be confusing. Avoid duplication, overlapping, and omissions.

Flag clearing stakes with white ribbon. In median areas or natural areas calling for the selective thinning or removal of trees, tie orange ribbon to trees to be removed and blue ribbon to trees to remain.

Grubbing limits do not require staking unless a special circumstance dictates. In general, grubbing is done in

FIELD NOTE RECORD (SKETCH GRID)

BOOK NO. ________ PAGE NO. ________

CONTRACT NO.	STATION	MILE/ LINE	C/S
L-4444	170+000 to 170+800		0883

STAKED BY	DATE	WORK STARTED	WORK COMPLETED
JVP		10-1-96	

CALCULATED BY	DATE	CHECKED BY	DATE	INSPECTOR'S SIGNATURE	DATE

distance from center line

grubbing limits

℄

grubbing limits

+000	27.1	8.5	18.6	35.4
+100	26.8	8.6	19.2	36.6
+200	24.4	7.9	17.7	34.1
+300	23.2	8.2	18.9	32.9
+400	23.2	8.5	19.8	32.9
+500	25.0	8.5	18.5	32.9
+600	29.0	8.5	18.3	32.9
+700	29.0	8.8	17.9	32.3
170+800	21.3	8.2	16.2	30.5

clearing limit

clearing limit

ITEM NO.	ITEM	GROUP NO.	DATE	UNIT	QUANTITY	RAMS NO.	BASIS OF MATERIAL ACCEPTANCE	CAPS ENTRY NO.	INTIALS POST	INTIALS CK	EST. NO.
2	clearing				m^2						
3	grubbing				m^2						

DOT 422-636 (Front) Revised 1/90

Field Notes for Clearing and Grubbing
Figure 14–1

excavation areas and in embankment areas where the fill height is 1.5 m or less.

Clearing should be completed one kilometer ahead of grading. Grubbing should be completed 300 m ahead of grading.

While staking for clearing and grubbing, the party chief must prepare a “Field Note Record” Form 422-636. See Figure 14-1. Complete the heading. In the sketch grid area, show center line, stations, and limits of clearing and grubbing. Include a north arrow. Do not try to draw to scale, as an exaggerated scale makes the note easier to read. Use a straightedge for straight lines. Remember to count isolated areas of less than 200 square meters as a full 200 square meters as described in Section 2.01 of the Standard Specifications.

See Figure 14-6 for an example of clearing and grubbing stakes.

Slope Staking

Slope stake and cross section or gather data for a digital terrain model as soon as clearing and grubbing is completed for an area.

Slope staking is the process of finding the point where the slope intercepts the natural ground. This point is referred to as the “catch point” or “slope catch.”

Cross sections can be taken while slope staking. Take the cross section shots at center line and the ground profile breaks along the line perpendicular to center line. The purpose is to determine actual quantities for excavation and embankment compaction payments to the contractor. On projects where clearing is not necessary the location cross sections may be adequate. The contract special provisions will indicate if this is the case.

The information needed to set slope stakes is obtained from the contract plans and the computer printout, “Template Elevation Listing.” You will need:

- Profile grade elevation.
- Traveled way cross slopes and lane widths.
- Shoulder slopes and widths.
- Side slopes, cut or fill.
- Ditch inslopes, backslopes, and depths.
- Total depth of surfacing materials.

A sample template elevation listing is shown at Figure 14-2.

To find the catch point, first obtain the distance and elevation of the subgrade shoulder for a fill section, or back of ditch (BOD) for a cut section. This information is found on the template elevation list. Next, determine the difference in elevation between the existing ground and the subgrade shoulder at that offset. Multiply the prescribed slope ratio by this difference in elevation to find the catch. If the ground is level, go to that distance and take another rod shot.

Note: Back of ditch elevation equals subgrade shoulder elevation.

Subgrade Shoulder

Station 26+820.00

ELEV	220.327		216.472		216.384		216.472		216.554		216.579
DIST	-21.38		-9.81		-9.55		-9.02		-4.88		-3.66
SLOPE		1:3		1:3		1:6		-.0200		-.0200	
ELEV	216.652		216.579		216.506		216.482		216.472		210.405
DIST	0.0		3.66		7.32		8.53		9.02		21.15
SLOPE		-.0200		-.0200		-.0200		-.0200		1:2	

Figure 14-2

In most cases, the ground will not be level. In a fill, if the ground slopes downward away from center line, the catch will be farther out. In a cut, if the ground slopes downward away from center line, the catch will be closer in.

The catch point is found by trial and error until the amount of cut or fill multiplied by the slope ratio plus the “add” distance agrees with the distance and elevation for the current rod shot.

The tolerance for slope staking is 30 mm horizontally and vertically.

Set a slope stake at the catch. Set an offset hub and stake to the catch. For fill sections the offset is 3 m beyond the catch and for cut sections the offset is 2 m beyond the catch.

When using a rod, level, and tape, the conversion from elevations to rod readings will expedite the staking.

From your H.I., determine the “grade rod” for subgrade shoulder (or back of ditch for a cut section).

This will be the rod reading if there is zero fill (or zero cut). After taking a ground shot at the subgrade shoulder distance (SGS), subtract the rod readings. The difference will be the height of fill or cut.

After two or three trial and error shots, the catch should be found.

After finding the catches for two to three stations, it is possible to align the rod on the previous stakes to eliminate much of the trial and error.

The following example is based on the template elevation list in Figure 14-2.

In cut sections, the back of ditch elevation/offset is used as the reference. This will prevent a contractor from under-cutting the subgrade by mistake.

H.I.	212.659	or	212.66	(millimeters may
SGS	<u>216.472</u>	or	216.47	be dropped.)
Grade Rod	3.813			

First trial shot at 9.02 meters (see Figure 14-3)

Rod reading	-0.85
Grade rod	<u>+3.81</u>
Fill	4.66

4.66 meters x 1:2 slope = 9.32 meters

9.32 meters + 9.02 meters = 18.34 meters

If the ground was level the slope would catch at 18.34 meters from center line, but in this case, the ground slopes down so the catch distance increases.

Trial shot #2 at 23.0 meters

Rod reading	-2.56
Grade rod	<u>+3.81</u>
Fill	6.37

(6.37 x 2) + 9.02 = 21.76

You are too far out.

Trial shot #3 at 21.2 meters

Rod reading	2.28
Grade rod	<u>+3.81</u>
Fill	6.09

(6.09 x 2) + 9.02 = 21.2 meters

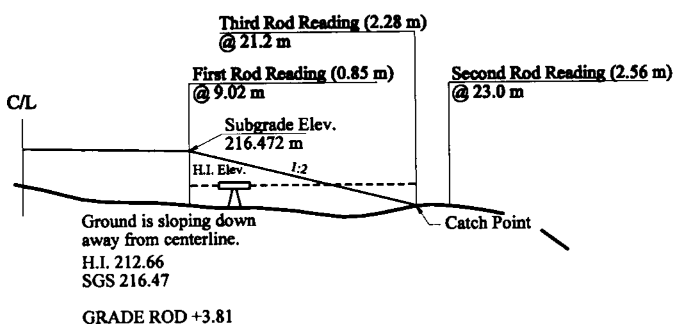


Figure 14-3

You have found the catch since the computed slope intercepts the ground at the elevation and distance of the current rod shot.

Using the same example template shown in Figure 14-2 for the left side we have:

Back of Ditch	
Elevation	216.47
H.I.	219.25

Grade Rod 2.78 m

1st Rod Reading @ 12.0 m 1.50 m (see Figure 14-4)

	2.78
	1.50
Cut	1.28 m

(1.28 x 3) + 9.81 = 13.65 m

The ground is going up which means more cut. Therefore, the 2nd shot needs to be farther out.

2nd Rod Reading 0.85 m
@ 15.0 m

Grade Rod	2.78
Rod Reading	0.85
Cut	1.93 m

(1.93 x 3) + 9.81 = 15.6

The ground is still getting higher but the catch should be within 1 meter.

3rd Rod Reading 0.61
@ 16.3

Grade Rod	2.78
Rod Reading	0.61
Cut	2.17

(2.17 x 3) + 9.81 = 16.32 m

The catch has been found. Set a slope stake at the catch point and an offset hub and stake for this point. See Figures 14-7 and 8 for example slope stakes. Obtain a rod reading and determine the cut to back of ditch elevation.

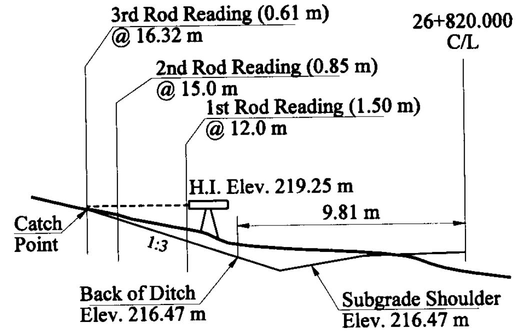
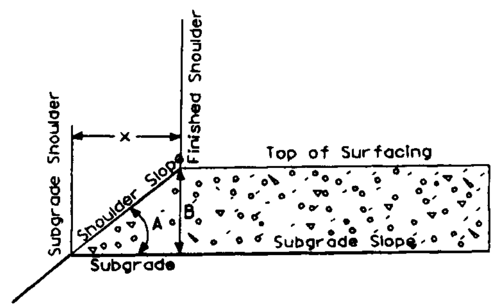


Figure 14-4

To compute the subgrade shoulder distance use the following formula and table. (See Figure 14-5.)



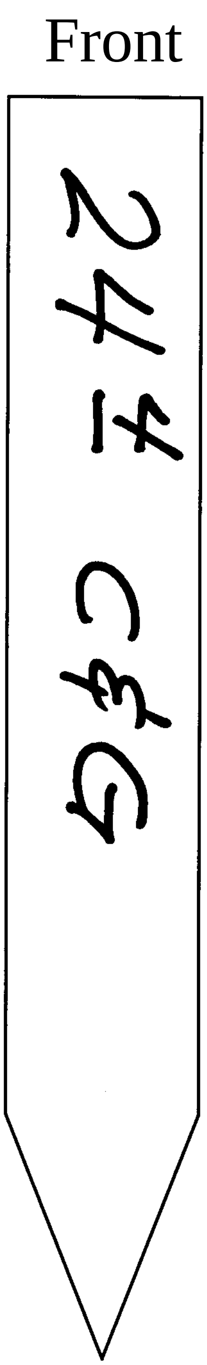
Distance From Finished Shoulder to Subgrade Shoulder and Slope Equivalents
Figure 14-5

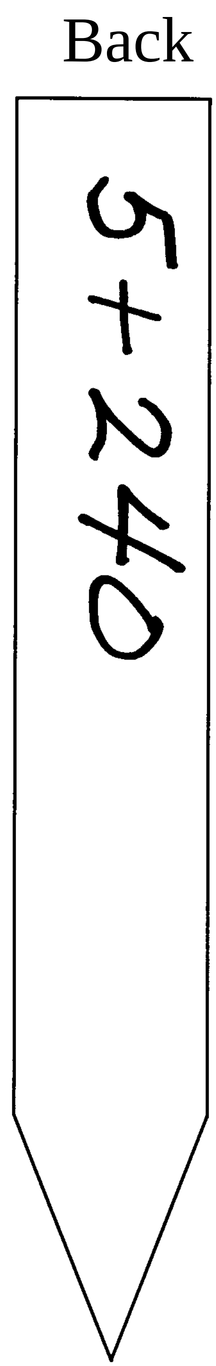
Equation: $x = \frac{100B}{A}$

A = Algebraic difference in % between shld. slope and subgrade slope
B = Depth of surfacing at finished shoulder
x = Distance from finished shld. to subgrade shld.

Shoulder Slope	Equivalent Rate of Grade	Equivalent Vertical Angle
1:1.5	66.67%	33°41'24"
1:1.75	57.14%	29°44'42"
1:2	50.00%	26°33'54"
1:2.5	40.00%	21°48'05"
1:3	33.33%	18°26'06"
1:4	25.00%	14°02'10"
1:5	20.00%	11°18'36"
1:6	16.67%	9°27'44"
1:8	12.50%	7°07'30"
1:10	10.00%	5°42'38"

Subgrade Slope	Equivalent Rate of Grade	Equivalent Vertical Angle
.020 / 1	2.00%	1°08'45"
.025 / 1	2.50%	1°25'56"
.030 / 1	3.00%	1°43'06"
.035 / 1	3.50%	2°00'16"
.040 / 1	4.00%	2°17'26"
.050 / 1	5.00%	2°51'45"





Use 1.2 m (4 ft.) Lath

Stake at 20 m
on Tangent Sections

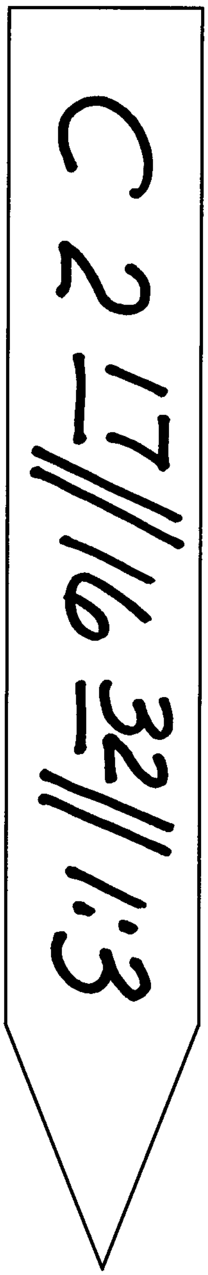
or at 10 m on Curves

or at 3 m on
Sharp Curves

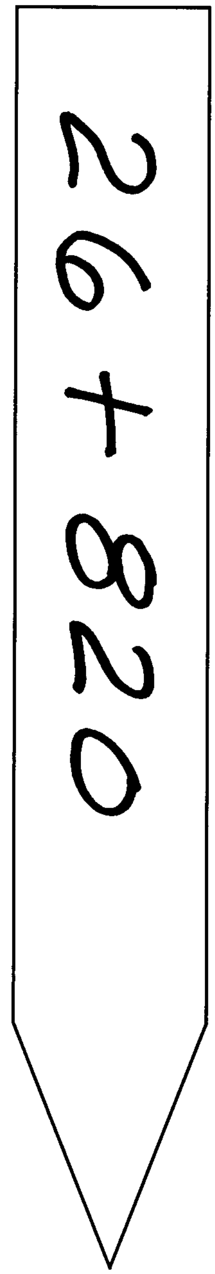
No Hub Necessary

Clearing and / or Grubbing Stakes
Figure 14-6

Front



Back



Cut is to Back of Ditch
Elevation and Distance

Use 1.2 m (4 ft.) Lath

Stake at 20 m
on Tangent Sections
or at 10 m on Curves
or at 3 m on Sharp Curves

No Hub Necessary

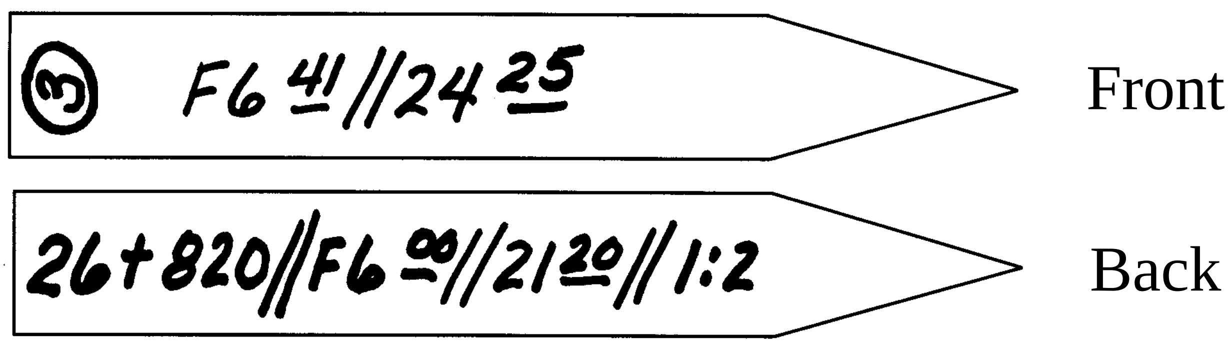
Slope Stakes
Figure 14-7

The offset stake is referenced to the subgrade shoulder/ back of ditch elevation.

On the back of the stake, write all the information that was on the slope stake.

It is important to place all stakes perpendicular to center line. This way when contractor's equipment removes the center line stakes, the center line station can easily be recovered by aligning with the slope stake opposite and taping the recorded distance to center line. A check distance to the opposite side will confirm the correct position.

Offset Stake for Fill Sections



Offset Stake for Cut Sections

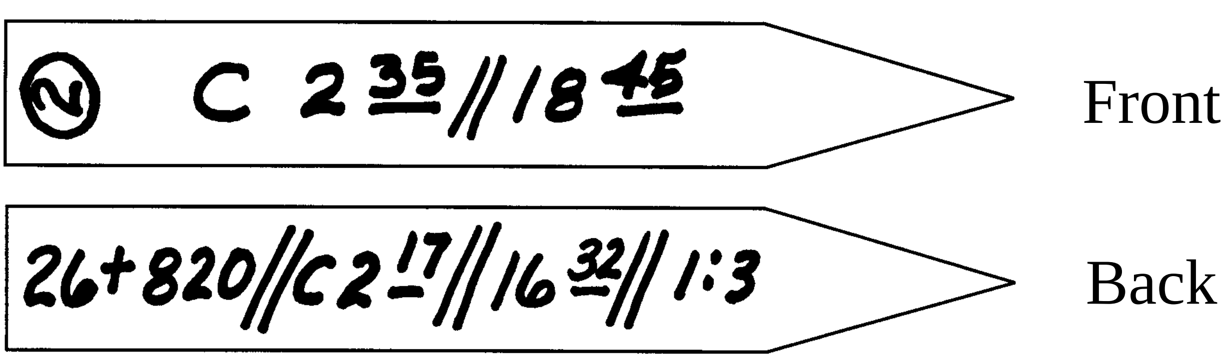


Figure 14-8

When “daylighting” isn’t workable because ditch water is draining toward your station, but the natural ground is still too low to obtain a full depth ditch, you may need to stake a ditch cut. The subgrade will be in a shallow fill, the ditch depth will remain the same related to subgrade shoulder, but the catch point will be lower than subgrade shoulder. In this case, slope stake using the bottom of ditch elevation as the reference. Label the stake as ditch cut (DC) with a fill to subgrade shoulder. (See Figure 14-9.) When the slopes change rates such as a 1:6 fill to a 1:2 fill, make as long a transition as practical. A change of one digit per station is acceptable.

Electronic Slope Staking

There are times when it is advantageous to use the Data collector with a “Roading” module to set out slope stakes.

Input roadway design templates, horizontal and vertical alignments, and superelevation rate data in the Data Collector before slope stake work can proceed.

Applicable situations would include new roadway designs with definite template ranges as opposed to roadway projects which match existing slopes. An advantage to slope staking this way is that the positions of the slope stakes are at right angles to the center line and this assures accurate quantities for payment.

In the latter case, it would be impractical to input unique templates for every station and therefore classical rod and level techniques would be appropriate.

Grade Control

After the roadbed is constructed as near as possible from the cut and fill stakes, the survey crew must set grade stakes for final finishing of the roadbed. There may be several sets of grade stakes required before the roadway is paved.

The first set of grade stakes usually required is for the subgrade. Stake subgrade before placement of base materials. The base may be crushed aggregates, cement treated or asphalt treated base of subgrade, selected rock material, etc.

The subgrade stakes are referred to as “bluetops.” The most common lengths used are 200 mm and 300 mm.

In most situations set bluetops every 10 m. In order for the contractor to grade the roadbed to its true cross section, place bluetops on each break in the slope.

Set bluetops on center line and subgrade shoulder. If a broken back is required, set a hub on this line. Also, if the width between hub rows is excessive, the contractor may want an intermediate row.

Do not set hubs too close together. A blade cannot work grades 2 or 3 m apart without destroying most of them. Do not set a “forest” of hubs in an intersection that the contractor could not possibly work between. It is best to set a few key hubs and return later if more hubs are required.

After the subgrade is completed, the base surfacing is placed at the prescribed depth. Then set hubs for base rock (“redtops”). Use the same procedure as for bluetops.

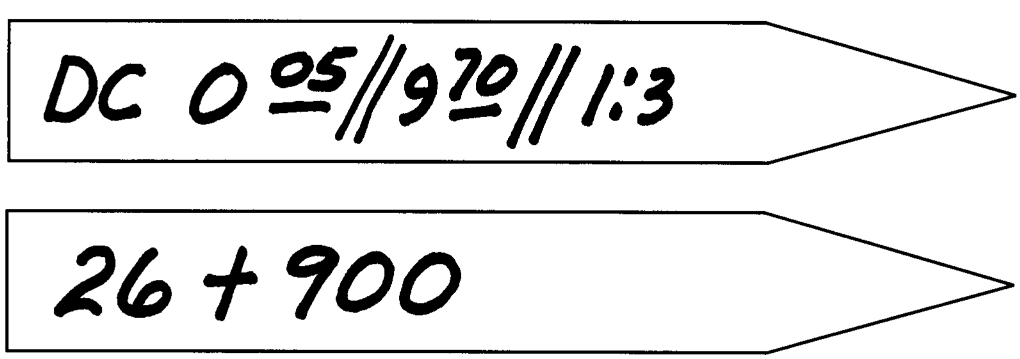


Figure 14-9

Finally, set “yellow tops” for the top course grade control.

Each row of shoulder hubs will be closer to the center line by the factor: slope times depth of surfacing course.

The finish required on roadway surfacing and subgrades shall ensure a final grade in as close conformity to the planned grade and cross section as is practicable, consistent with the type of material being placed. See the contract and Chapter 1 of the *Construction Manual* for hub depth requirements and tolerance for each course.

Set hubs for subgrade 15 mm below “grade.” Set hubs for surfacing at “grade.”

Equipment

The equipment required to set grade stakes is as follows: Level, level rod (Philadelphia or Lenker), sledge hammers (5.5 kg), pick, shovel. A small pruning saw can be useful and a frost pin is also often needed.

The ideal rod to use is the Lenker or self-reducing type. Grade rods do not have to be computed as with the Philadelphia rod. The Lenker enables you to read the elevation directly on the rod with no computations required. For example, the TBM elevation is 127.68. The rod is set on the TBM and the rodman adjusts the tape until the levelman reads 7.68 and then locks the tape in place. If the elevation of the bluetop should be 125.15, then the hub is driven until the levelman reads 5.15 on the rod and no computations are required.

If you use a Philadelphia rod however, the following computations are required: The TBM elevation is 127.68. The levelman reads 5.06. The HI is then 132.74. The grade elevation (125.15) is subtracted from the HI (132.74) to yield a grade rod of 7.59. Thus, when the hub is driven so the levelman reads 7.59, the hub is at grade. Grade rods must be computed for each hub set.

A 5.5 kg hammer is a good all around size for driving the hubs. A lighter hammer requires too many blows to drive a hub into a well compacted grade which is tiring and increases chances the hub will be shattered. The heavier the hammer the better. Skill with a hammer is the controlling factor in the speed at which the crew can work.

A pick and shovel are necessary tools when the grade is a little high. Digging below grade before driving the hub makes the hub easier to drive. If the grade is consistently high or low, halt operations and inform the inspector or foreman. If the grade is suspected of not being close, spot check it before setting up operation. Painting, on the ground, the amount high or low may save time in the long run.

A small pruning saw can save much time and work in some situations. In a very hard grade often the hub will go so far and then bind up. The hub will shatter before getting to grade. If the hub is solid, saw it off at grade. The alternatives are: keep pounding and if the hub shatters set another; or remove the hub and drive a frost pin to provide a hole for the hub.

The hammered end of the frost pin should periodically be cut off. Do not use a mushroomed tool.

Crew

The ideal size crew for bluetopping is usually 5 people: a levelman and two teams to drive hubs (a rodman and a hammerman). The rodman operates the rod and carries the stake bag with extra hubs and small tools. The hammerman drives the hub, sets the guard stake, and brings along a pick or shovel if required.

Procedure

The procedure for efficient setting of grade stakes is as follows:

1. (TBM’s set every 150 m are recommended on construction projects.)

2. Run the center line.

3. Go through and set a stake at each point you intend to bluetop.

4. Drive through with the survey truck and throw out hubs at each point. 300 mm hubs are used the most. Select the lengths appropriate for the soil conditions.

5. Set up the level, set the Lenker rod on the bench mark, and read the elevation. Set the rod to the elevation and check it.

6. Each rodman sets the rod next to the stake and calls out the station.

7. The levelman reads the rod and indicates how much up or down to grade. He usually calls out the amount of cut or fill.

8. The hammerman drives the hub, stopping above the indicated grade. The rodman checks and then the hammerman drives the hub to grade with light blows. Don’t overdo it. It is easier to tap a hub down a few millimeters than it is to pull one up. A good set of hand signals known to all may be useful.

   Turn when you reach the limits of your level. Check all rods into a TBM as often as possible. After obtaining a new H.I., check the last hub set.

P:HSM14

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Drainage

As in all survey work, it is easy to transpose numbers or make other blunders, but it is hard to pay for the results. Therefore, some other qualified person should check all computations and spot check instrument readings before leaving the site.

Culverts

A culvert is an opening in the embankment which allows water to pass from one side to the other. Culverts are placed in valleys that would otherwise be dammed by the highway embankment.

Culverts may be concrete or metal pipe, pipe arches, or concrete box culverts. The amount of water passing through and the height of the fill determine the size and type of culvert to be installed.

If the culvert is to be constructed for a flowing stream, a channel change is usually required. The culvert is constructed on the new channel alignment and the stream is then diverted through it.

If the channel is dry at the time of construction, the contractor may be required to partially build the embankment before placing pipe.

To lay out culvert installations, perform the following steps:

1. Consult the contract plans for station and offset for ends of the installation.

   You may be required to field fit culverts. If so, find the slope catch in the channel bottom for each end of the culvert.

   Check the templates to be sure widening for guardrail has been included if necessary.

2. Set a hub and tack at the indicated positions for each end of the culvert.
3. Measure distance between hubs.
4. Set a parallel offset line at a distance convenient for the contractor. Usually 3 m is adequate.
5. Beginning at the downstream end, set and station hubs along the offset line at 10-m intervals. The beginning hub should be station 0+001 or greater.
6. At the downstream end, set a second hub to ensure proper positioning of the first section of pipe.
7. Obtain elevations on all offset hubs and corresponding ground elevations at the center line of the pipe. Record on *Field Note Record for Drainage*, WSDOT Form 422-637 (Figure 15-2).
8. When the trench will be excavated to a depth of 1.2 meters or more, obtain elevations at the horizontal limits of the trench.
9. Compute the flowline grade of the culvert for each offset hub. Subtract from hub elevation. Record on the form.

10. Mark and place stakes at the hubs, recording the station, offset, code number, and cut. See Figure 15-1.
11. Check all computations and check all stakes for accuracy in recording.
12. Complete the sketch on form 422-637 (Figure 15-2a) along with other required data (Figure 15-2b) and submit to your supervisor.

Sewers

Sewers are a closed system of watertight pipes that generally begin and end in some sort of drainage structure.

Storm sewers, manholes, or catch basins are located to allow water in or out of the system and provide access for cleanout. Manholes are usually spaced at a maximum of 100 m. Catch basins are spaced often enough to drain the roadway.

Sanitary sewers will have manholes for maintenance but no other openings. In sewer design, the crowns of all pipes should coincide at the center of the manhole. Therefore, water running through a small pipe into a larger pipe at a manhole will fall by the difference in pipe size.

On the drainage plan sheets of the contract plans you will find a circled number and a line drawn to each drainage structure. The plan sheet number with the circled number are the “structure note” or “code.”

The drainage profile sheets show the station, offset, flowline grade, and top of grate elevations for each installation. The top of grate elevation should be at the center and is usually at the pavement elevation for manholes, and 25 mm below the pavement elevation for catch basins and grate inlets. The grates should be set on the same slope as the pavement.

Grades are critical, especially for sanitary sewers. Therefore, pay close attention to elevations.

The “structure notes” section of the contract plans tabulate the lengths, size, type of pipe, appurtenances, and any special note for each installation.

In laying out sewers, the following steps are taken:

1. Study the plans, special provisions, structure note sheets, standard specifications and appropriate standard drawings before starting. This is most important. In studying the system you are staking, be sure to consider the whole system, not just the area you are working in. You may pick up an error on the plans before it gets constructed.

   Make sure that the back edges of catch basins will be in the curb line and that manhole lids are not in the curb line.

2. Establish the locations at the center of manholes, catch basins or any other connections by setting a guinea.

   A hub and tack serves no purpose since it will get dug out. Set an offset hub at the same offset distance as for the pipe. Then set a second offset hub in line with the first. This will allow the contractor and inspector to be assured of accurate placement of the structure.

3. Compare the plan layout against the ground layout. If you suspect an error in the plans or if something looks out of place, advise your supervisor or project inspector. Do not make changes without approval.

4. Set RP hubs for an offset line parallel to the pipe at 10 m intervals. Station the hubs using 0+000 at the center of the manhole or catch basin at the lower end of the pipe.

   The RPs need to be offset enough to allow pipe laying and digging equipment room to work along the trench but not so far away that they are difficult to transfer to the trench. Usually 3 to 6 m is a good offset distance, depending on the depth and size of pipe. Consult with the pipe laying contractor and agree on the offset distance.

5. Obtain elevations on the offset hubs and ground at the center line of the pipe. Record the elevations on WSDOT Form 422-637 (Figure 15-2).

6. Compute the flowline grade of the sewer at each RP hub. Subtract from hub elevation and record on the form.

7. Mark and place stakes at the hubs, recording the drainage code number, station, offset, and cut. See Figure 15–1.

8. Check all computations and check all stakes for accuracy in recording.

9. Complete the Field Note Record for Drainage (Figure 15-2). One sheet is required for each drainage structure note or code. The form must show a simple plan view and profile along with a north arrow and center line ties. The back of the sheet (Figure 15-2b) will show the pipe stations at 10 m intervals, flowline grades, ground elevations at the center line of the pipe, the elevations of the offset hubs, and the cuts from the offset hub to pipe flowline grade. The inspector will complete the quantity calculations.

When completed, submit the form to your supervisor.

As in all survey work, it is easy to transpose numbers or make other blunders, but it is hard to pay for the results. Therefore, some other qualified person should check all computations and spot check instrument readings before leaving the site.

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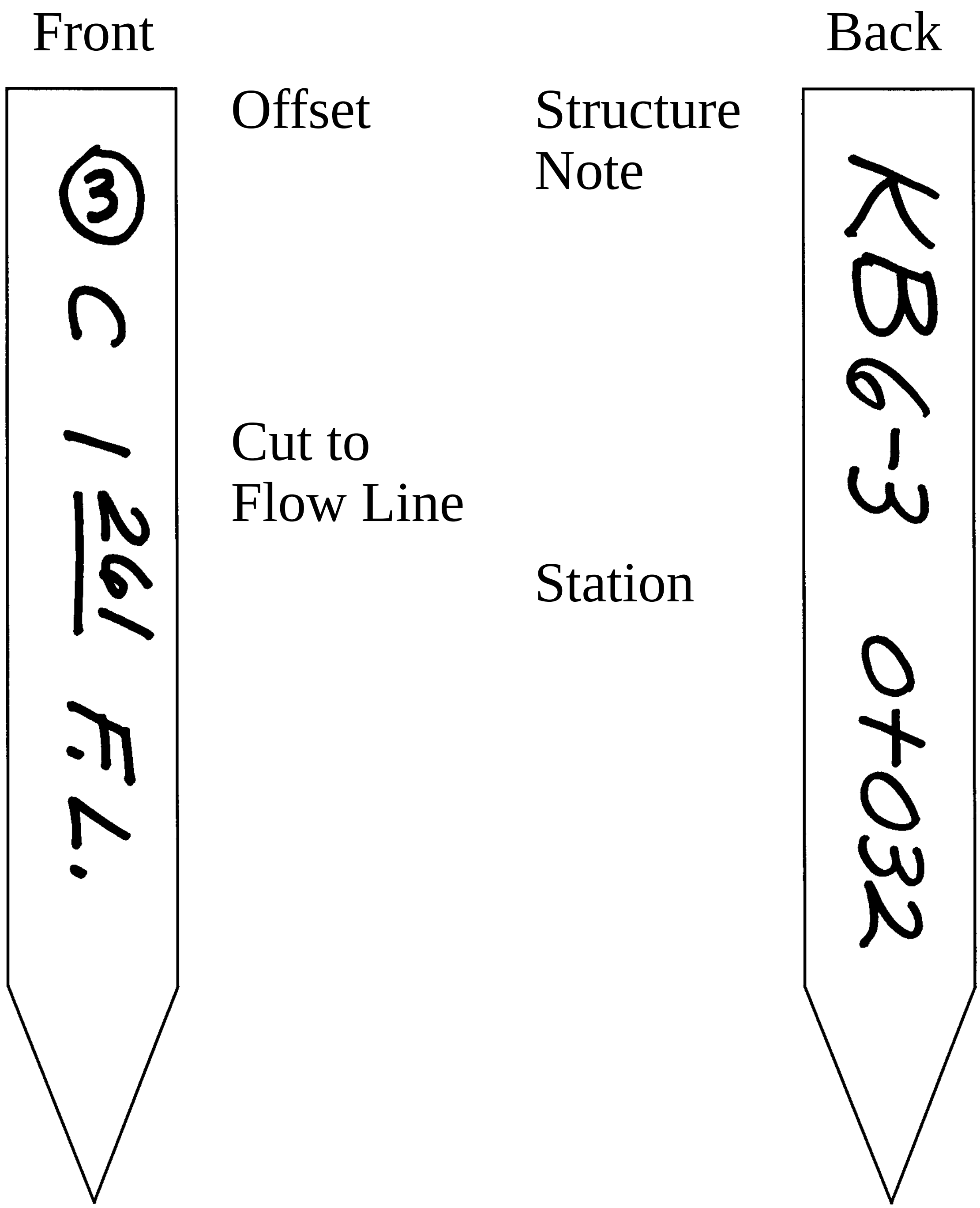


Stakes for Drainage
Figure 15–1

Washington State Department of Transportation

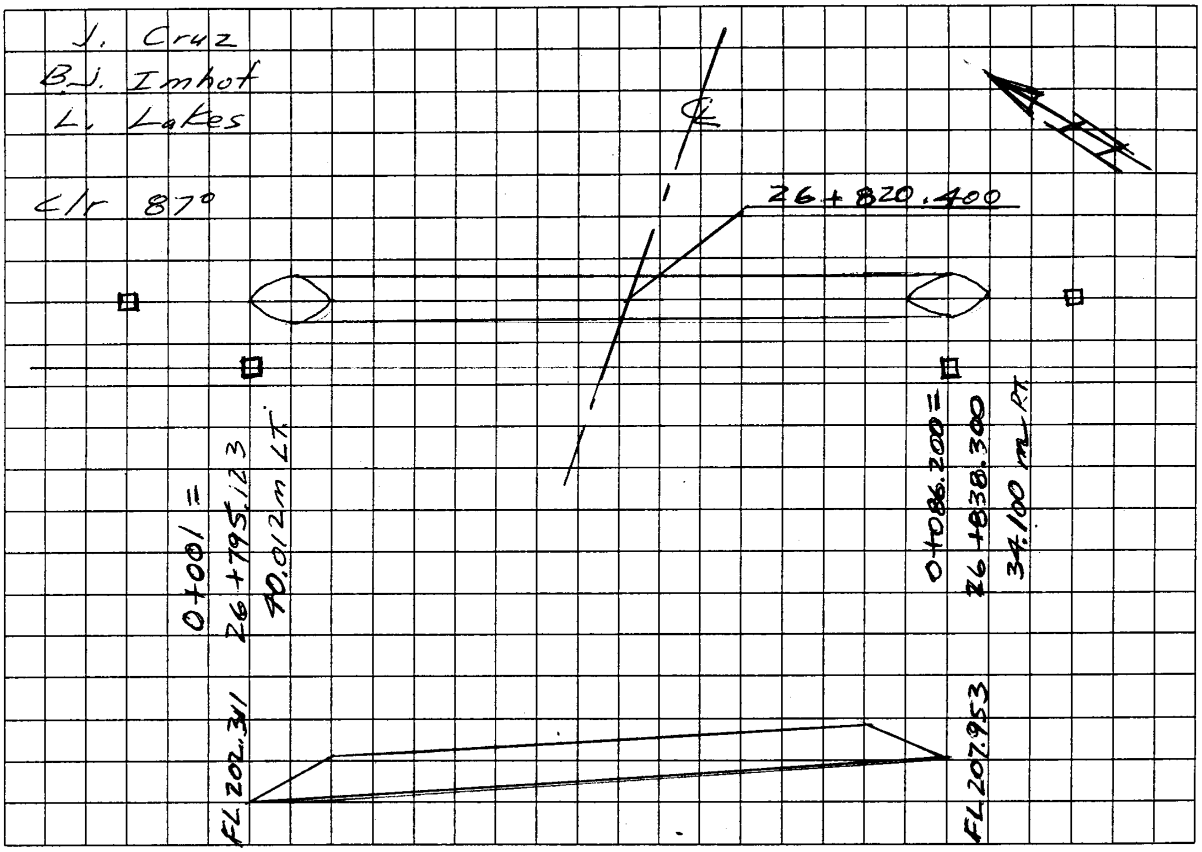
FIELD NOTE RECORD FOR DRAINAGE

BOOK NO. ________ PAGE NO. ________

CONTRACT NO.	STATION	LINE	C/S	CODE NO.
1996	26+820.4			PD 5-21

STAKED BY	DATE	WORK STARTED	WORK COMPLETED
JVP	8-25-96		

CALCULATED BY	DATE	CHECKED BY	DATE	INSPECTOR'S SIGNATURE	DATE



ITEM NO.	ITEM	GROUP NO.	DATE	UNIT	QUANTITY	RAMS NO.	BASIS OF MATERIAL ACCEPTANCE	CAPS ENTRY NO.	INTIALS POST	INTIALS CK	EST. NO.
28	Tr 2 Steel Culvert Pipe 1.5mm Thick 122 cm Diam.										
77	Structure Excav. Cl. B Incl. Haul										
80	Shoring or Extra Excavation Cl. B										
85	Gravel Backfill For Pipe Bedding										
86	Gravel Backfill For Foundation										

DOT Form 422-637 (Front)
Revised 9/95

Field Note Record for Drainage
Figure 15–2a

BOOK NO. ____________ PAGE NO. __________

STRUCTURE EXCAVATION

(PIPE STRUCTURE EXCAVATION WIDTH = 2.6 m)

STATION	0.0662 m/m FLOW LINE GRADE	ORIGINAL GROUND	SUB-GRADE	CENTERLINE CUT		6 m OFFSET HUB	OFFSET CUT-F.L.	REMARKS
				FLOW LINE	BOTTOM DITCH			
0+000		202.06						
0+001	202.310	202.31				202.496	0.186	
0+010	202.906	204.60				204.811	1.905	
0+020	203.570	204.56				204.905	1.335	
0+030	204.230	205.72				205.649	1.419	
0+040	204.892	206.83				205.888	0.996	
0+050	205.554	207.10				206.907	1.353	
0+060	206.216	206.85				207.100	0.884	
0+070	206.878	207.81				207.302	0.424	
0+080	207.540	206.99				207.923	0.383	
0+086.215	207.950	207.95				208.141	0.191	

REMARKS

DOT Form 422-637 (Back)
Revised 9/95

Structure Excavation
Figure 15–2b

16

Structure Layout

This chapter will explain the procedures to be followed for surveying and staking structures.

Review Section 1-5.6 of the *Construction Manual* before proceeding.

Review the plans, specifications, and special provisions.

Horizontal Control

For structure layout, a horizontal control network should be established using second order, class II procedures. Place control points in strategic locations so that any point within the bridge site can be set from at least two control points. The control points must be substantial enough to remain in place and undisturbed for the duration of the bridge construction. A rebar and cap set in concrete could be used for these control points.

The next step is to establish the structure center line . Sometimes this center line differs from the line used to construct the roadway. Study the plans carefully to determine the correct line. This is not always plainly marked and it is easy to overlook some variation in the alignment. Resolve any problems before setting stakes.

Run the center line (make sure it closes within the site location) and all other controls that may be pertinent to the structure.

Check distances across streams, highways, or other obstructions by use of an approved electronic measuring device (EDM).

Never rely on any existing station to establish pier or bridge footing locations without checking it first. Always double check all distances. Errors in locating the footing might necessitate extensive revision in the design of the structure or removal of the incorrectly located foundation.

Staking

Stake pier locations in accordance with the footing layout included in the plans. After the piers have been staked, stand back and “eyeball” the entire layout, if possible, to determine if it looks correct. The depth of footing compared to the ground line, cut or fill slopes, or stream bed should be checked. Profiles should be taken along the center line of each pier or bent and at all corners for use in excavation calculations.

Vertical Control

Vertical control should be set after the horizontal control. Set temporary bench marks (TBM) in readily accessible locations where they will also be safe during construction. Extra TBMs are advisable to ensure survival.

The vertical control requirement for major structures is second order class II and for minor structures is third order.

When setting TBMs for structures, remember that structures are very susceptible to settlement. Not only do the piers settle but the ground in the area of the piers can also settle. Therefore, it is essential to set a control TBM outside the area of influence so that it can be used to monitor TBMs near the structure. Settlement may occur on the day of the pour or more than a year later, so it is something that requires close attention. Settlement of several centimeters is common in some concrete structures and the inspector and P.E. should be notified when it is detected.

TBMs should be located at each end of the structure. If the structure is very long, it may be necessary to set TBMs in intermediate locations along the structure. Consider locating TBM in the vicinity of each pier.

Layout and References

Stake the piers/footings at the locations shown in the plans. These may be staked directly by station and offset from the center line or from the control points established previously. Stake reference points in line with the center of the pier or footing to ensure recovery of the pier after excavation.

Consider the following concepts when setting R.P.s.

Consider the length of time that the point must remain in a precise, fixed position. It should be resistant to outside forces such as traffic and frost heave. Hubs are susceptible to heave if not planted deeply enough and will weather rapidly if shattered. If not driven straight, small diameter rebar has a tendency to straighten during weather cycles. If a rock is driven along the side of a rebar to get it on line, it will only remain in that position until the next heavy rain or frost.

A 1:5 ratio from RP to deck elevation is sufficient width on structures other than lids.

The reference points should be placed so they are clear of other construction features and so they will not be affected by ground movements caused by large excavations or embankments. Set adequate references so that if some are lost the pier can still be easily reestablished. References are a very critical item in the layout of the structure. A little extra time spent placing good references can save time throughout the life of the project.

Checking Layout

The layout of the structure and the references should be independently checked by either a different survey crew or by the same crew using a different control point. All survey notes should also be checked by another person.

Whenever possible, distances shall be determined by direct measurement.

The contractor will excavate for the footings. (The party chief as well as all crew members may only enter properly shored excavations. It is a safety violation to do otherwise.) The bottom of the excavation must be blue topped for grade, and the form corner hubs set.

After the piers or footings have been cast, the reference hubs can be used to center and plumb the column forms and then the pier cap forms.

Superstructure

As the contractor’s work progresses to the superstructure, the survey work also continues.

For box girder bridges, the bottom deck, diaphragms, webs, and exterior walls are aligned on the plywood decking. For precast girder bridges, points are set for precise placement of the girders.

When working on structures, safety procedures for work above ground or water must be known and observed. Due to the heights involved as well as heavy materials being lifted overhead and proximity of water or traffic, the potential for injury is high. It is the responsibilty of the party chief to ensure that items such as handrails are in place before the crew begins work.

The contractor will construct the forms for the roadway deck. The roadway will extend outward of the exterior walls or girders. This portion of the bridge is called a soffitt or overhang. The decking forms are supported by either steel brackets or bracing against the falsework below. On the overhang decking, the line marking the edge of the roadway deck must be laid out. The elevation at each bracket/brace is observed and the soffitt is adjusted until the decking is set at its predetermined elevation. This procedure is known as grading the overhang. The brackets/ bracing are usually spaced at 1.2 m along the entire length of the bridge.

Only approved levels in proper adjustment and approved rods may be used for determining elevations and grades for structures. Total station instruments are not designed with the tolerances required for the vertical dimension. Remember to check the TBMs from the control benches after major concrete pours and after placement of large quantities of rebar.

The interior decking forms are usually adjusted to grade after placement of the rebar so that some of the "crush" is taken up. The girders or web walls will have rebars protruding vertically from the concrete. Grade marks can be filed into the bars for each bay and adjustments are made by string lining. The screed rails are set up and the crew will grade them at each support bracket.

Critical areas to watch for in deck construction are errors built into structures due to expansion caused by temperature changes. In steel structures, camber built into truss spans can increase more than 25 mm on a hot day. When doing this type of work, a point can be set at mid span and then monitored throughout the day to determine what if any adjustment to apply to the layout grades. This is less obvious on concrete spans so it is easier to miss. This is probably the greatest cause of dispute when state crews are required to check grades after they are set and adjusted by the contractor. A typical scenario has the grades set in the morning when it is cool and checked in the afternoon by a different survey crew.

The final survey operation will be performed after the falsework has been released. This is the grading of the top of the traffic barrier. First, the face of barrier line is established and deck elevations observed at each joint in the forms. The deck profile is plotted and top of barrier grades are determined. "Low spots" can be filled but any "high spots" in the deck will essentially be a controlling factor. The chamfered grade strip is nailed to the back form and concrete is poured to match the grade.

Documentation

Field Note Records will be required for payment of the contractor. Keep a field book record of observations made, sketches, horizontal and vertical control monuments, and H.I.s for each grading operation. Also keep computer calculations and other papers. A separate field book for each structure is recommended. These records should be turned over to the office engineer for safekeeping upon completion of the project.

P:HSM16

17

Miscellaneous Construction Surveying

Pit Site / Quarry Site, Stockpiles, Sundry Site, and Reclamation

Pit Areas

One of the many tasks required of survey crews is to gather field data of pit site areas for the purpose of generating quantities for the state to pay contractors.

With current surveying practices, this is done by gathering Digital Terrain Model (DTM) data of the pit site area before any work is done (original ground) and then to gather new DTM data after work has been performed (remeasure).

The contractor may request a remeasure of the pit site at a time when the work is getting close to plan quantities.

The procedure is as follows:

1. Gather crew together and discuss strategy for collecting data.

2. Set a minimum of two control points to establish a set up station and a backsight. Assumed coordinates and elevations may be used for this type of work since the object is a volume.

3. Set additional control points as necessary, to be able to “topog” the entire pit site area. Locate all control points where they will not be disturbed by the work.

4. Set up a new “topo” job in the data collector with a note which includes the pit site number, date, job name, and the names of crew members.

5. Set up and orient total station.

6. Select the collimation program in the data collector.

7. Collimate on an object that is about the same distance away as the data collecting shots will be.

8. Record collimation and select the topography program in the data collector.

9. Begin to gather data for DTM. It is very helpful to mark each point where a shot is taken with a paint spot so you can visually see areas where shots are needed.

10. Use WSDOT standard survey codes.

11. Download, edit, backup, and create DTM.

For remeasures, create a new job in the data collector and follow the aforementioned procedures taking care to pick up any terrain changes from the original ground (first DTM).

In CEAL, use the Volumes Between Two Surfaces command to generate pay quantities. This command compares the DTM created with the first job in the data collector (original ground) and a DTM created when remeasuring the pit site.

Classical cross section methods may be used instead of DTM methods. The procedures are as follows:

1. Establish a base line and reference it in a safe location so that it can be replaced after the material has been removed. Fifteen meter intervals are usual but more may be required if the ground is very irregular.

2. Cross section the ground prior to removal of any material.

3. Establish temporary bench marks (TBM) for vertical control. Set 2 or 3 TBMs around the pit for backup.

4. On completion of the removal, or at any time an estimate is required, reestablish the base line and cross section the area. This will be the remeasure line on your notes.

Stockpiling

The volume for pay quantity will be determined by computing the volume between the original ground surface and the stockpile surface using Digital Terrain Model (DTM) techniques or by cross sectioning. (See the Standard Specifications Section 3-02.)

Before the survey crew begins establishing areas for stockpile stakes, they should first study the contract plans to determine the following:

- Number of stockpiles required for various aggregates.
- Which aggregates are to be surveyed for pay quantity.
- Quantity of aggregates required in each stockpile.

An on-site review of the stockpile area with the project inspector and contractor will determine where the best stockpile locations are in relationship to the various plant operations.

1. Stockpile areas will be located to ensure easy access by trucks and loading equipment.

2. Do not establish stockpiles under power lines.

3. Maintain sufficient distance between the various stockpiles to prevent mixing the various classes of materials.

The procedure for staking stockpile areas is as follows:

4. Find out how much area is available for the stockpiles. Often a scale drawing of the area will be needed.

5. From the proposed planned quantities, determine suitable dimensions for piles that will best fit into the available areas. (If stockpiles are for maintenance, check with the maintenance foreman for the best locations for their use.)

6. Lay out control points, base lines, and all corners of each pile and indicate materials to be piled at each located area.

7. Follow the procedures given for DTM technique for pit areas or, follow steps 8 through 12 below for cross sectioning.

8. Place stations along the base line at 5-m intervals and at any points where irregular breaks in the ground require extra stations for accurate measurements.

9. Cross section the ground from the stations established along each base line. Take rod shots at 8-m intervals and where ground line irregularities require extra rod shots for accurate measurement.

10. Set base line references in protected areas.

11. Set a TBM in a safe place. Assumed elevations can be established if an actual bench mark is not handy. Use the same bench mark for all stockpiles being constructed in the immediate area.

12. After the stockpile is complete, remeasure the pile and compute the quantity.

Sundry Site And Reclamation Plans

Refer to the *Plans Preparation Manual* and the *Standard Plans for Road, Bridge and Municipal Construction* for survey requirements.

Erosion control

Erosion control is to preserve the erodable surfaces of our highway projects. Steep slopes are the most susceptible to erosion. Covering with topsoil then planting grass, sodding, bark mulch, plants, and trees are used for erosion control.

Measurement and payment are by area, volume, or actual count of plants, trees, etc. For items paid by area such as seeding, mulching, and fertilizing, the crew must measure the area to be covered.

For example: in a quadrant of an interchange, start by laying out a base line. The base line may be parallel or skewed to the highway, ramp, or crossroad. If the base line is on a skew, tie the ends to one of the control center lines.

After the base line is laid out and stationed, measure to the edges of the planting area perpendicular to the base line at 15-m intervals or at the breaks. These measurements are made directly on the ground instead of level chaining.

A Field Note Record must be prepared showing the base line and center line ties and measurements to the edge of the planting area. The inspector then computes the areas for payment.

Guide Post

The locations for guide posts are shown in the contract plans. Check the standard plans and specifications for the distance from the edge of the pavement as it may vary in different areas. Check with the project engineer because maintenance may have some special requests as to the location of the guide posts.

In staking guide posts, usually all that is required is a stake at the location of each guide post. Sometimes a paint mark on the edge of the pavement will be sufficient. Consult with the contractor who will be doing the work.

More information can be found in the *Design Manual* Chapter 830 or the *Manual on Uniform Traffic Control Devices for Streets and Highway* (MUTCD).

Guardrail

Study the plans, specifications and standard plans specific to the project before beginning. This is especially important with guardrail as the design details change frequently.

Use the face of rail and top of rail as the horizontal and vertical references for guardrail.

Consult the contractor for the type of reference line he needs. He may want the face of rail staked, the center of post, back of the post, and so on. He will usually want the center of the bolt or the top of the post referenced for grade.

Resist the tendency to overstake guardrail. Normally, only the guardrail ends (flares and parabolas) and the tapers are staked. The following will apply in most cases.

- Don’t stake every post. Just stake the beginning and ending post of the straight run.
- In most situations, a reference hub every 15 m is enough.
- On sharp horizontal or vertical curves, you may need a hub every 10 m.
- On tight radius curves, you may need hubs at 3 m.
- A hub and tack is usually used to mark line and grade.
- Only stake every post in special areas where the situation requires exact placement.
- If the contractor wants the face of the rail staked, it is not necessary to keep the hubs between the posts.
- If the contractor wants the center or back of the post staked, locate the hubs between posts.

Do not level chain as the rail has a finite length which follows the slope of the road.

Fencing

The locations for fencing are given in the contract plans or the special provisions.

Consult the project inspector and the contractor for the type of reference line.

The following will apply in most cases:

- Stake the pull points, gate post, corner post, and end posts.
- For all types of fence, a change in alignment of 0.6 m tangent offset, or more, for the next post is considered a corner.

More information can be found in *Design Manual* Chapter 1460.

Illumination and Traffic Signals

First study the plans, specifications, and standard drawings. The locations are given in the contract plans. A visual check of the site is needed to assure that the location is suitable.

See the plans for the location and the height of the luminaire. A cross section is necessary at that section of roadway to determine the base elevation.

Stake the center of the base for line and grade. The stake for the center will be destroyed during excavation and must, therefore, be referenced. Locate the reference points outside the excavation area, but near enough to be useful to the contractor. See Figure 17-1 for offset hub locations. Show the offset distance, cut or fill to the top of the luminaire base, and stationing on the stakes (Figure 17-2).

Stake the conduit runs according to the plan or as directed by the project inspector. A paint line will generally be adequate for the conduit, but consult the project inspector and the contractor for their needs.

Stake and reference the junction boxes. See the plans for the location.

Service cabinets must be located and referenced to line and grade.

When staking illumination, good communication is needed with the project inspector. The survey crew must be aware of his needs and aware of any changes that are necessary.

Good communication between the crew and inspector will aid the crew in determining what is needed.

More information can be found in *Design Manual* Chapters 840 and 850.

Signing

Steel Sign Supports

Study the plans, specifications, and standard drawings. The following procedure usually can be followed:

1. Locate the site of the sign. It will be given on the plans but you will have to check it for visibility.
2. Take a cross section at the sign location. Include the edge of the traveled way in the section.
3. Set hubs at the footing locations with references to the top of the footings.

With this information the exact post length can be computed, and the sign can be installed later. Unless your reference hubs are lost, no other staking is usually required.

On sign bridges and large cantilevered signs that have electric power to them, you will have to also locate and stake the conduits. Follow the plans and try to locate the conduits outside the pavement area if possible.

Miscellaneous Signing

The locations for these facilities are given in the contract.

All that is required for wood sign posts is the location by station and the offset. The required heights and other details are given on the plans and are the responsibility of the contractor.

More information can be found in *Design Manual* Chapter 830 or the *Manual on Uniform Traffic Control Devices for Streets and Highway* (MUTCD).

Pavement Marking

Review the contract plans and special provisions, *Standard Specifications*, and *Standard Plans* prior to laying out pavement marking.

Prior to layout, meet with the contractor to determine the intervals required for layout marks. Once layout is complete, the contractor completes preliminary spotting prior to pavement marking.

Center line is generally used as the control points for pavement marking although use of an offset is acceptable. Lay out all the pavement marking from the same control to maintain correct spacing and lane widths. Paint marks are typically used when establishing control and for layout.

Center Line

Mark center line with paint.

Curves

Use a transit and set the curve points or use a 300 ft or longer cord, eyeball in a curve, and paint over it. By doing the tangents first, it will make the transitions to curve look better when you eyeball in a curve.

Concrete Curb

When staking concrete curb, check the plans carefully to determine the correct location and grade.

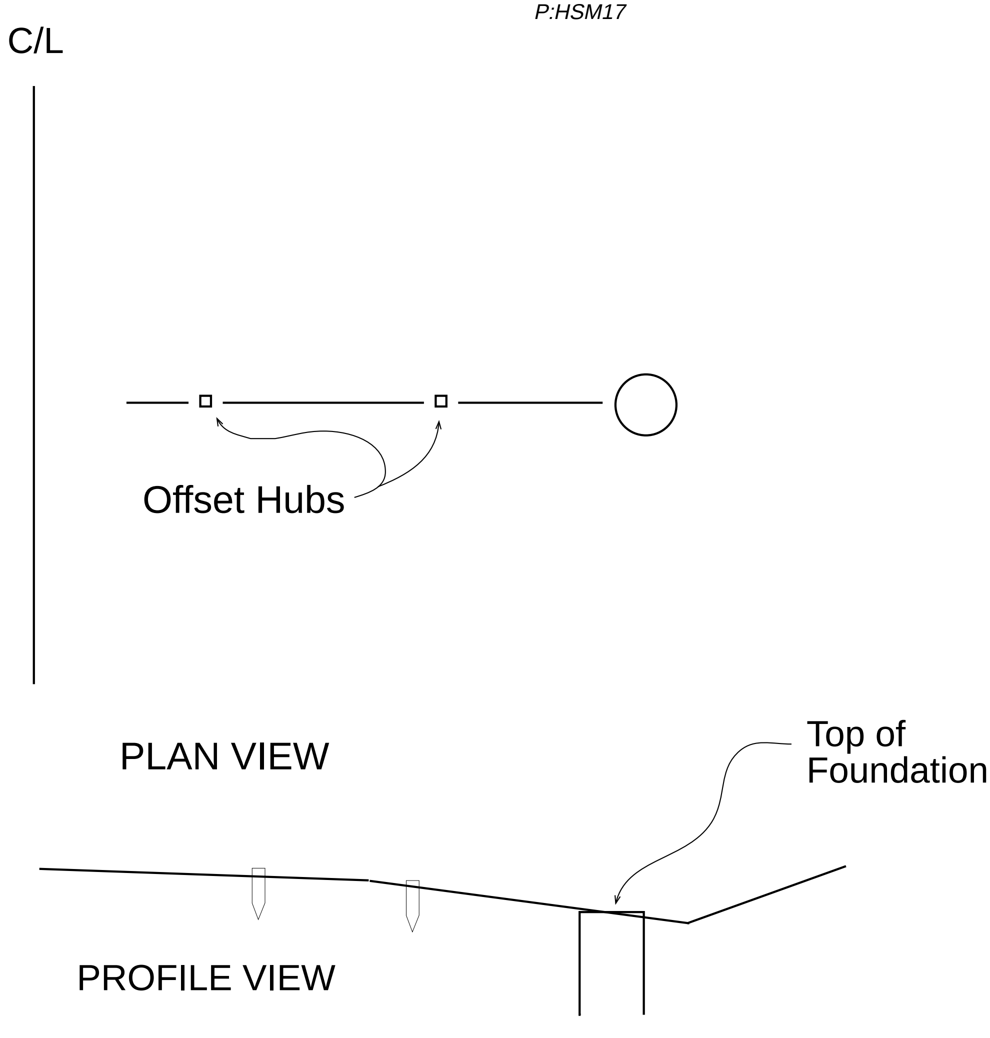
Since the face of the curb is battered and the back of the curb is plumb, the best control point for line and grade is the top and back of curb.

Set hubs and tacks at 10 m intervals on a 1 m offset line to the back of curb. On radius curves of 15 m or less, hubs and tacks should be spaced at 3 m or less as needed.

Run levels on the hubs and determine the difference in grade from the hubs to the top of the curb. Prepare stakes showing the difference as a cut or fill and the offset. Flagging on the stakes is not necessary. See Figure 17-3.

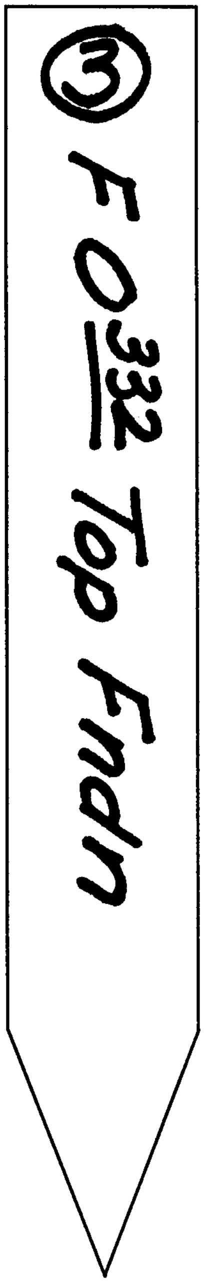
A Field Note Record must be prepared showing the plan view of the curb line along with the length staked.

P:HSM17

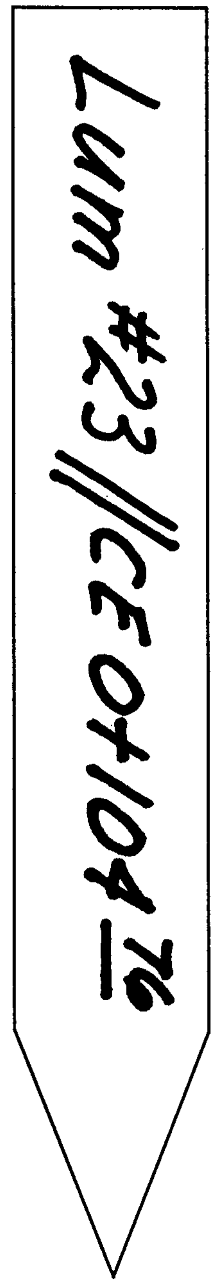


Staking Illumination and Traffic Signals
Figure 17-1

Front



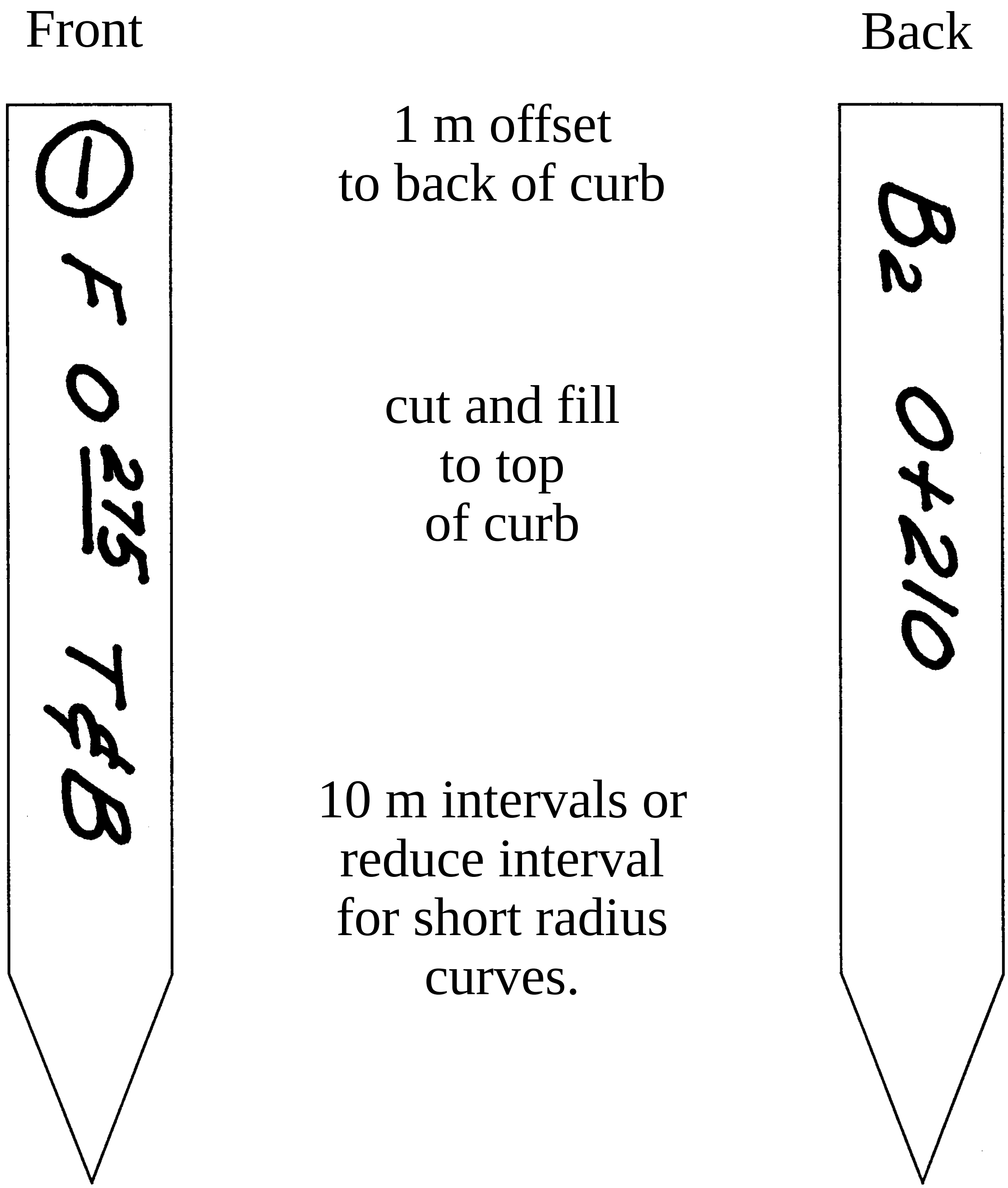
Back



3 m offset to center of base
cut and fill to top
of concrete base.

Set 2 hubs and stakes
in line.

Stakes for Foundations for Illumination,
Signs, and Signal Systems
Figure 17-2



Stakes for Curb
Figure 17-3

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Post Construction Surveys

The post construction survey serves several purposes:

1. It establishes coordinates and elevations for the monumentation set by the contract.

2. It provides the necessary data for making a record of monumentation.

3. It establishes a permanent base line from which the right of way line and right of way base line can be reestablished.

4. It establishes a base line for future projects in the area.

5. It enables WSDOT to comply with RCW 58.09 and 58.20.

6. It provides a permanent record of how the survey measurements to determine monument location were performed.

Monuments

Monuments include both horizontal and vertical markers.

A horizontal marker monuments a horizontal survey position.

A bench mark monuments a vertical position.

The same monument may serve as both a horizontal marker and as a bench mark.

See the contract for type and location of monuments.

Procedure

Determine during the location phase where monuments are to be placed and how monuments are to be designated and marked. The following criteria should be followed:

- Monument location should be accessible after construction.
- Monuments should be intervisible and not liable to have the line of sight blocked by brush, trees, or future construction.
- Monuments should be set back from the traveled roadway as far as possible and still meet above requirements.

The procedure is:

1. Designate location of each monument by station and offset on the contract plans. The location shall not be changed by construction personnel without approval by the location personnel.
2. Include as contract items the installation of monument case and cover (in paved areas), poured-in-place concrete (in unpaved areas) or drilled hole (in rocky areas).
3. If the actual setting of the monument is done by a WSDOT crew, cement the disk inside the pipe, set in the freshly poured concrete or cement into the drilled hole, as applicable. If the monument is to be set by the contractor, furnish WSDOT disks to the contractor. Stamp each disk with the predetermined monument marking before it is set.

4. Mark the location on the ground where the contractor is to install the monument case and pipe, pour the concrete, or drill a hole. The location of the control point shall be within 3 mm of the specified location as shown on the contract plans. If the contractor installed the monument, verify that it was set to this specification.

5. After completion of construction, perform a third-order traverse and a second-order class II leveling through the monuments.

6. Prepare a Record of Monumentation showing the location of each monument set by its station and offset referred to the right of way base line, together with its state plane coordinates and file it with the county engineer. Send copies to the Department of Natural Resources and the Geographic Services Branch. A copy will also be kept in the region files. The Record of Monumentation shall be signed and sealed by the engineer responsible for the survey.

7. If the surveying of the contract is being done as a contract item, the contractor is responsible for engaging a licensed land surveyor to perform the leveling and traversing through the monuments and to prepare and file the Record of Monumentation. The contractor shall provide a copy of the Record of Monumentation to the Project Engineer for the region files, together with a copy of the field measurements and calculations.

P:HSM18